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DECEMBER, 1952. **Vol. XLVII, No. 12**

FORTY-SEVENTH YEAR OF PUBLICATION

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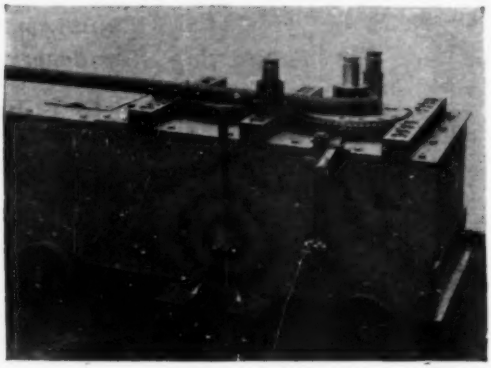

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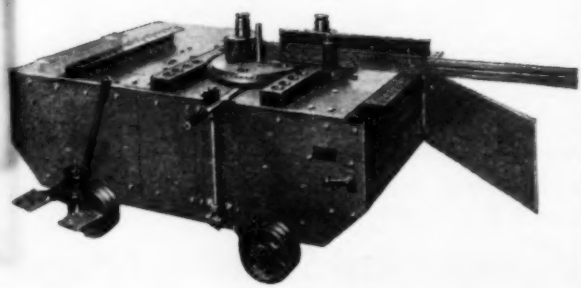

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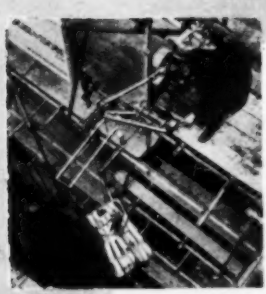

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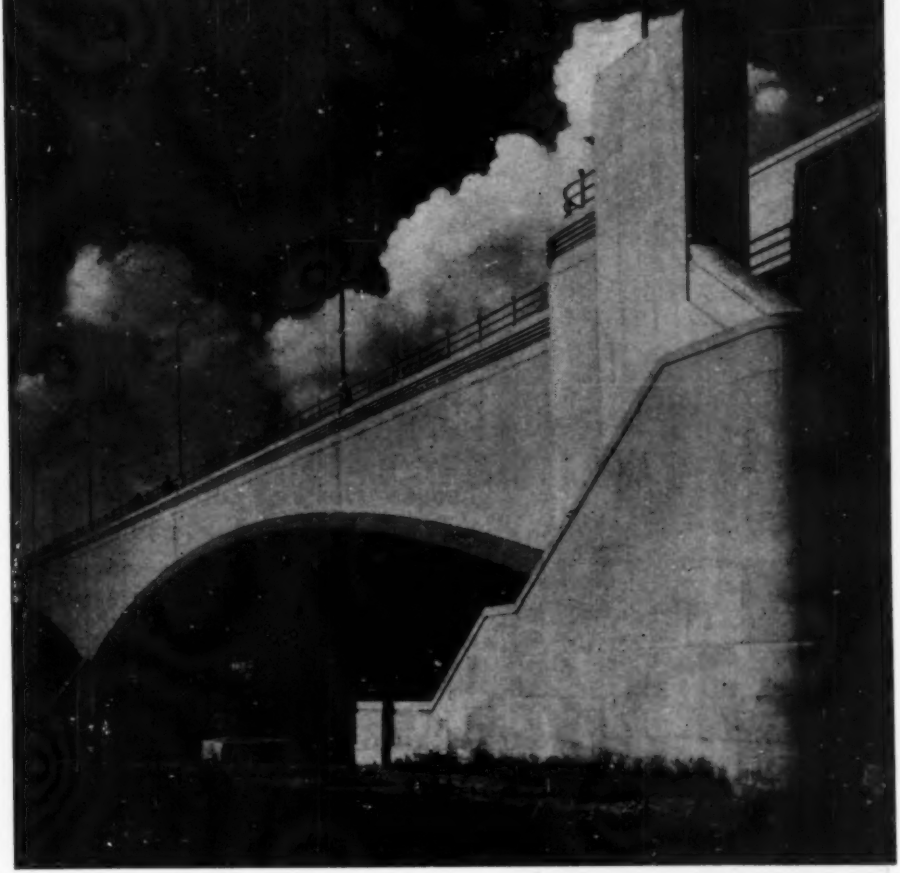

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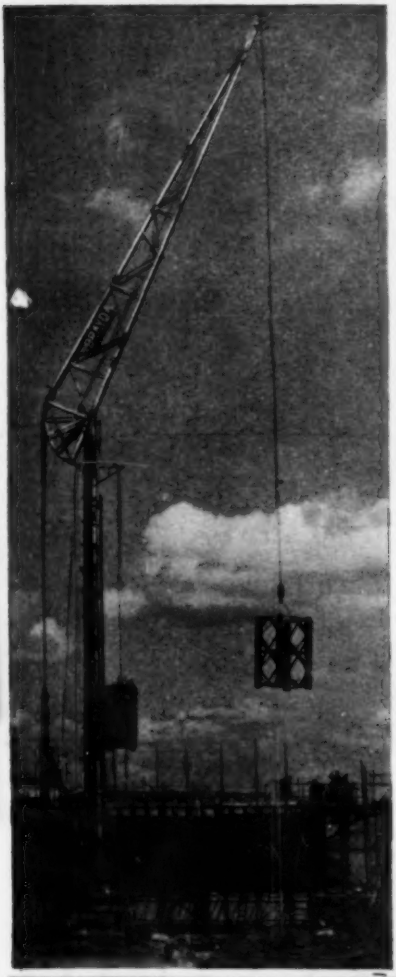

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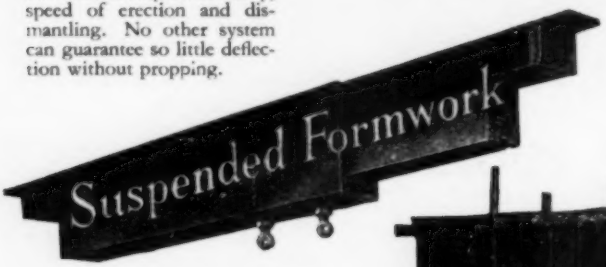

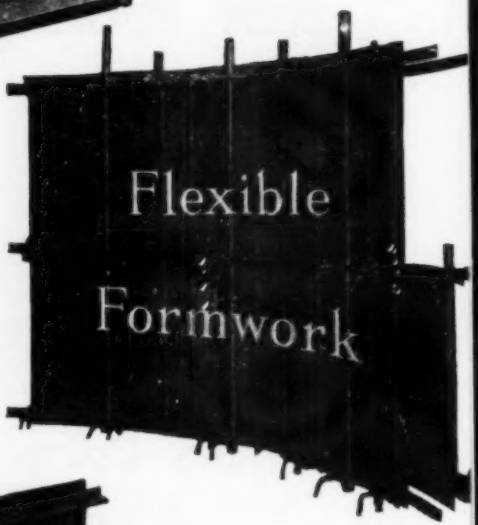

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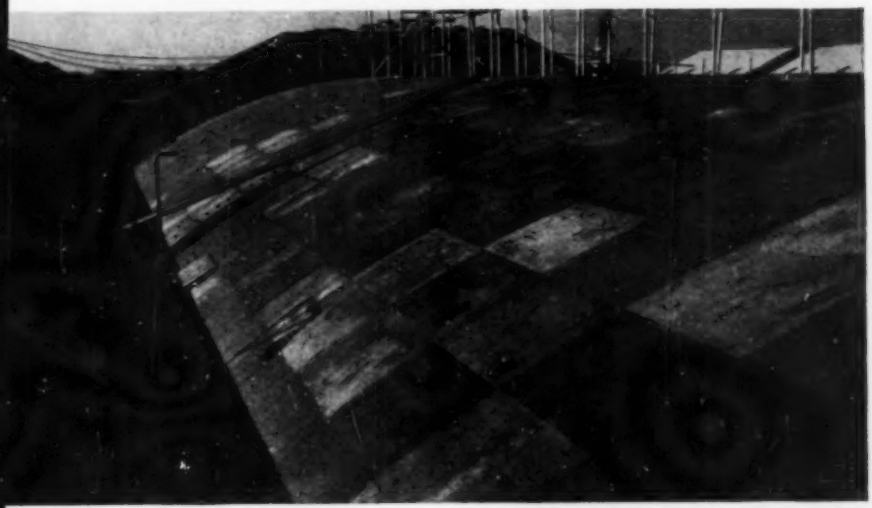
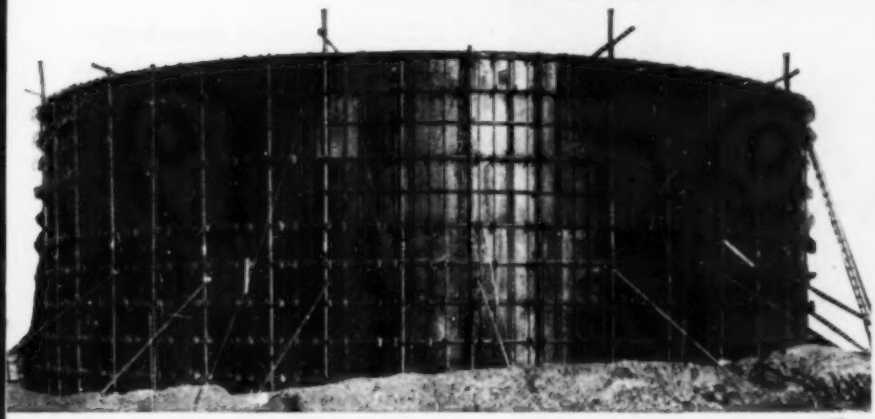


The illustration shows Flexible Formwork in position to receive reinforcing.



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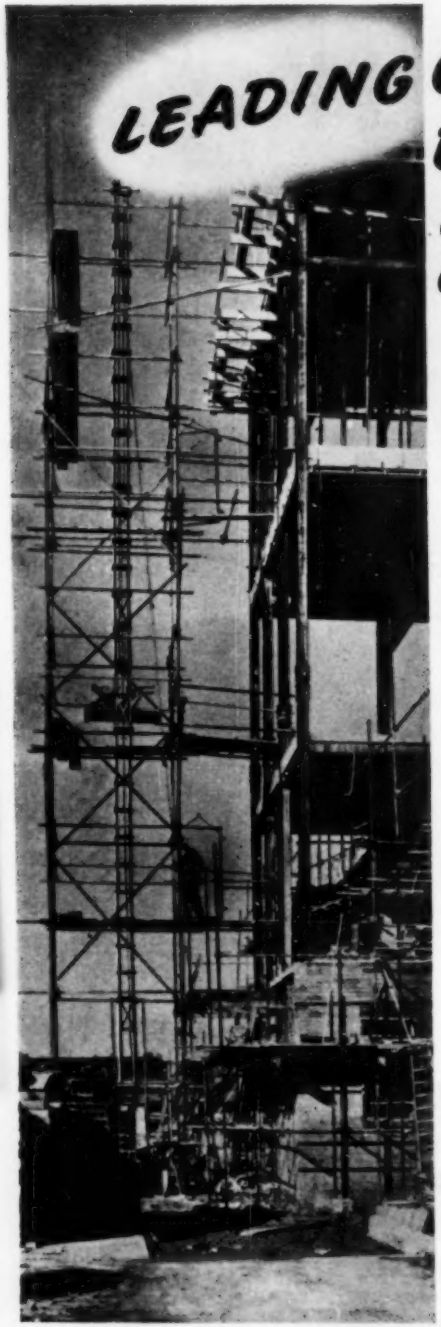

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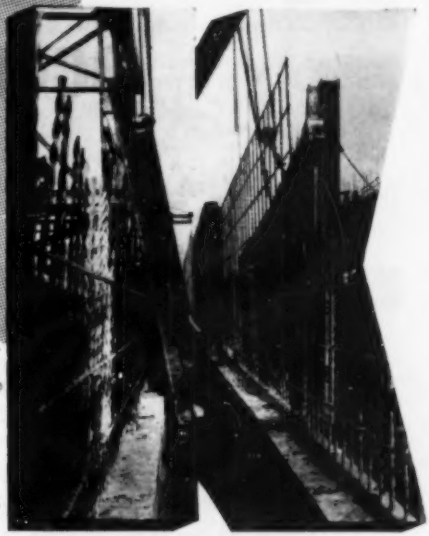

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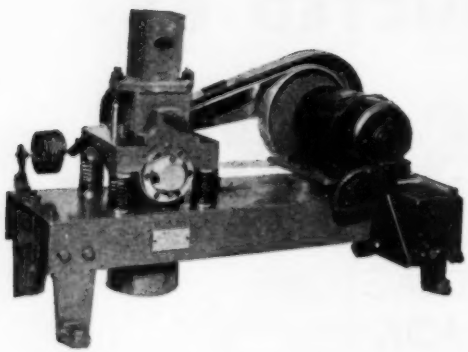

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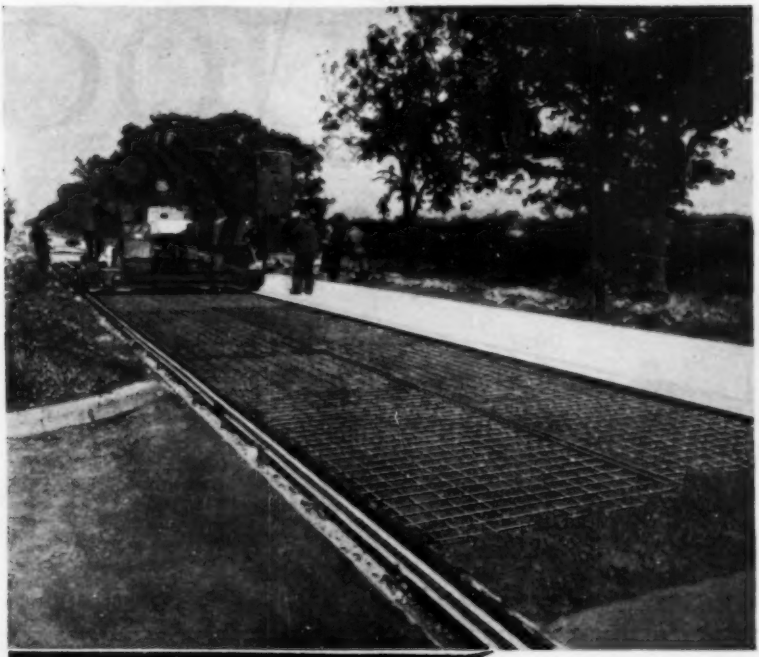

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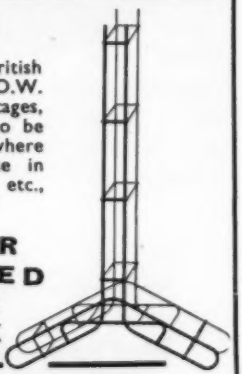

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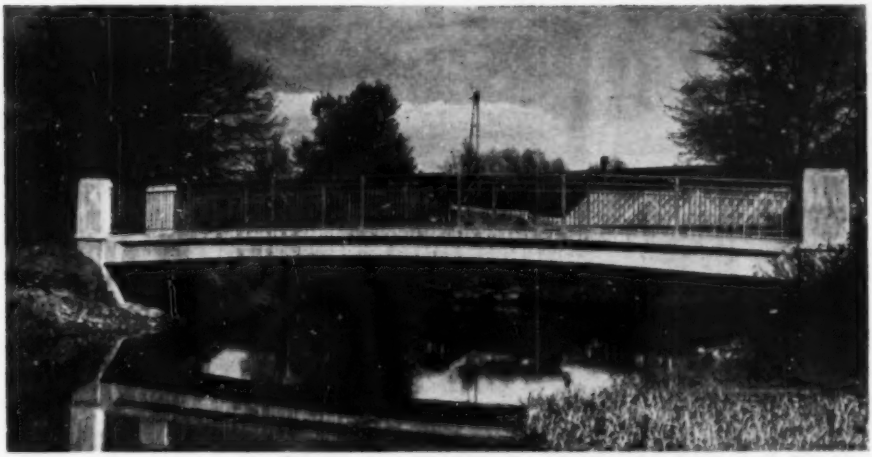

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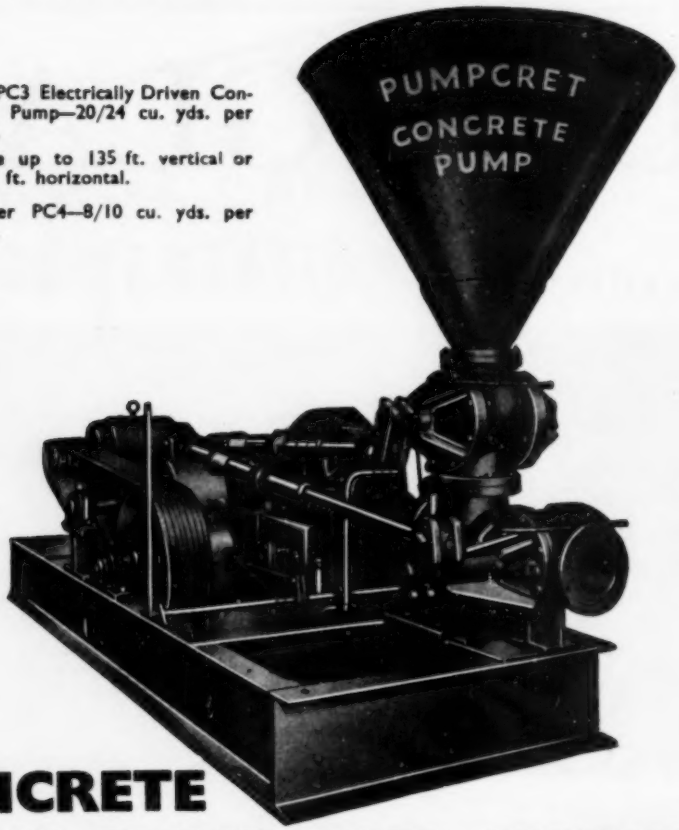

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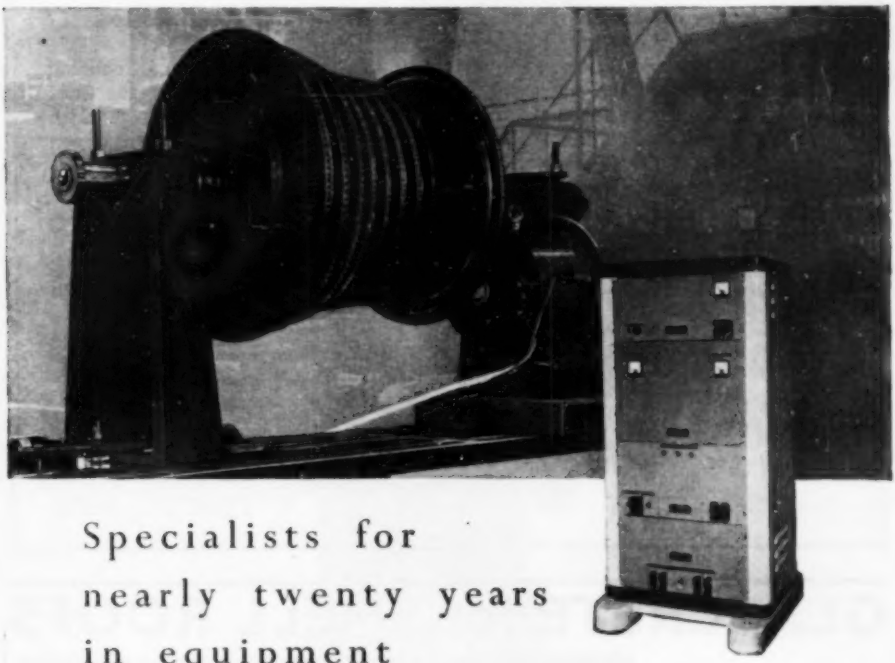

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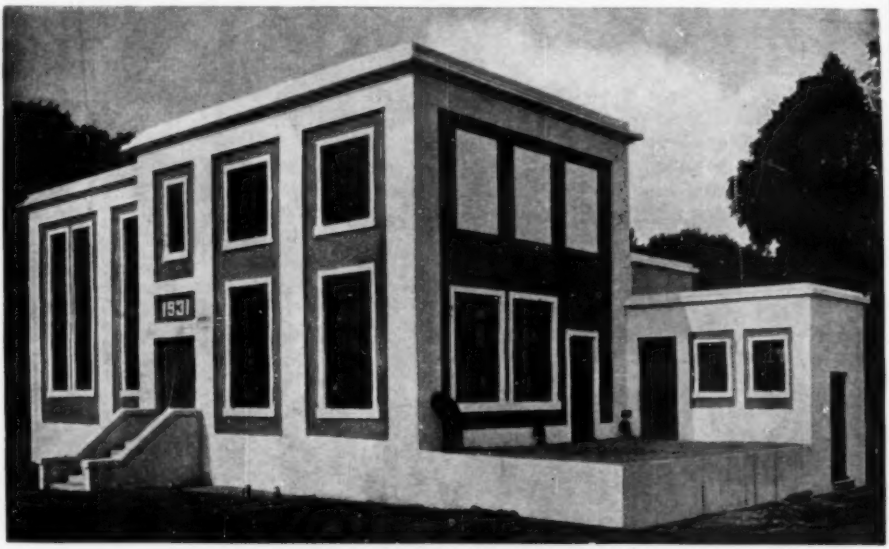

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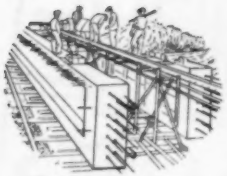

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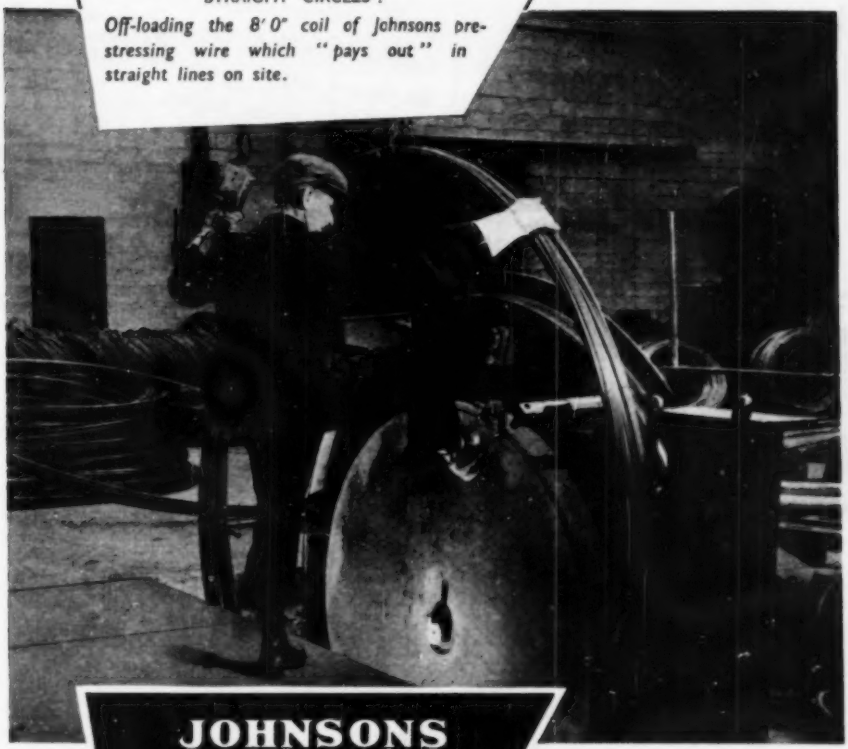

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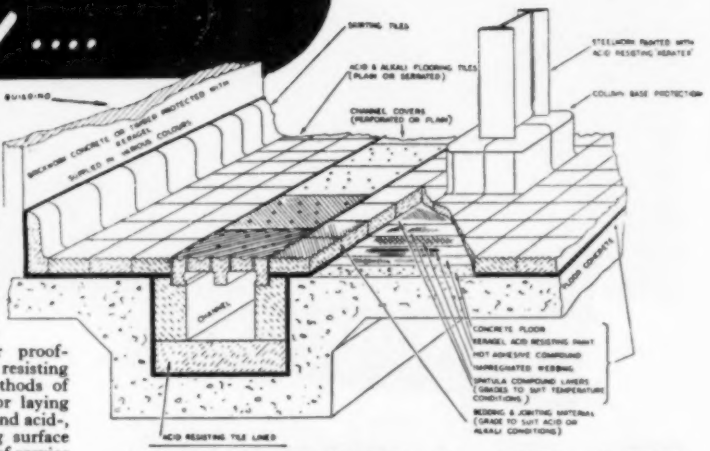
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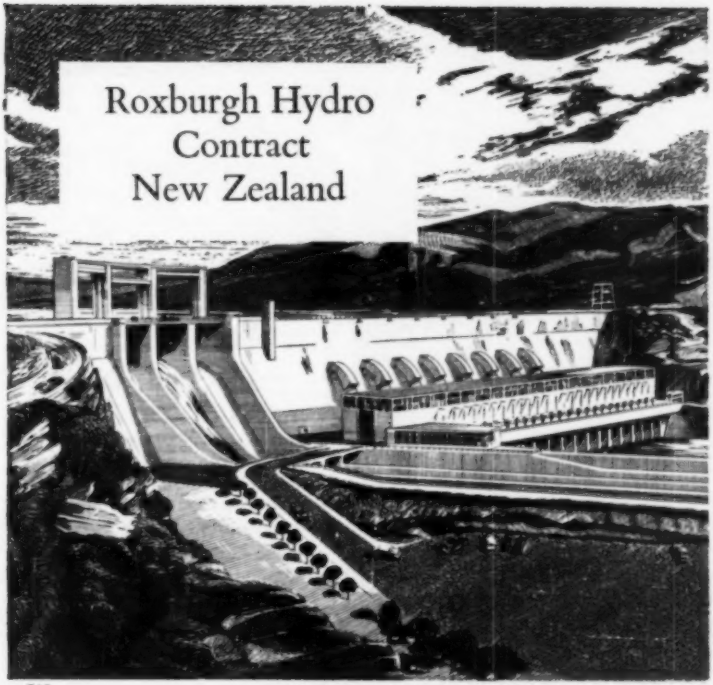

C62. *Based on an original drawing by C. H. Haggis, F.R.S.A., F.I.B.D.*

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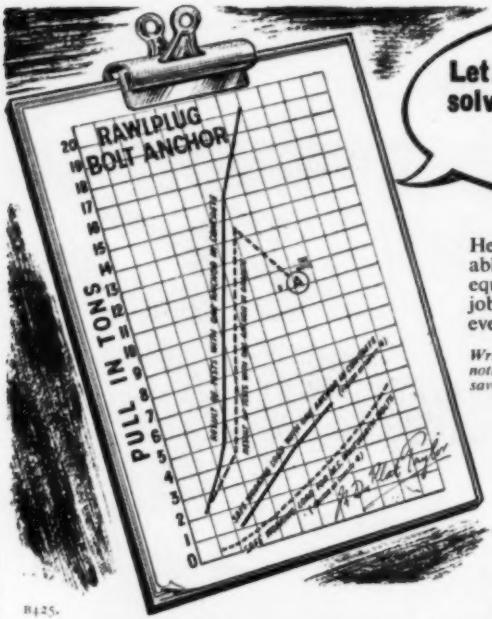

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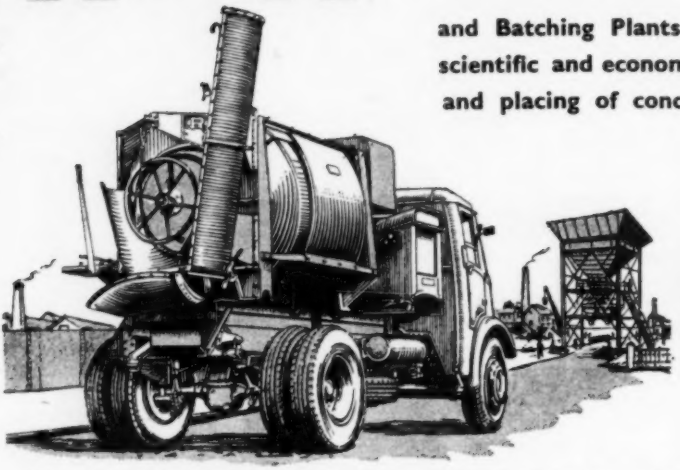

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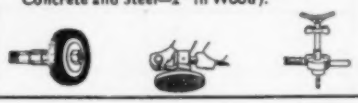

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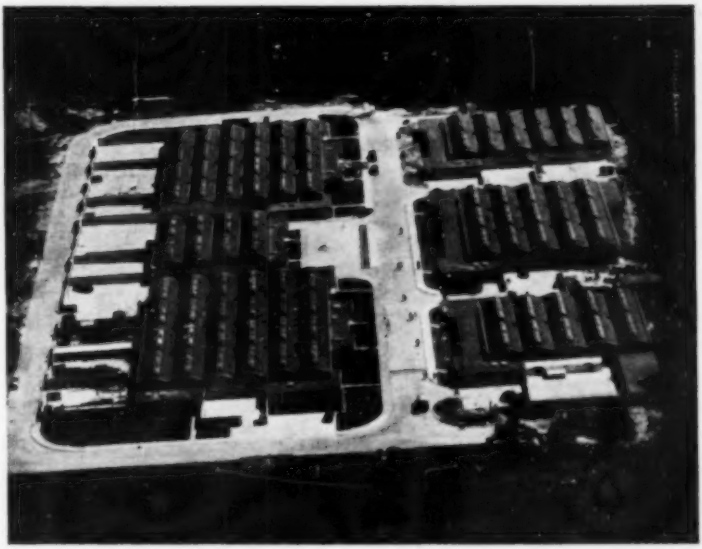

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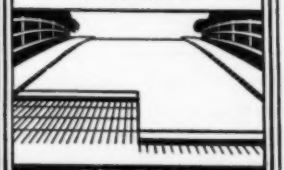

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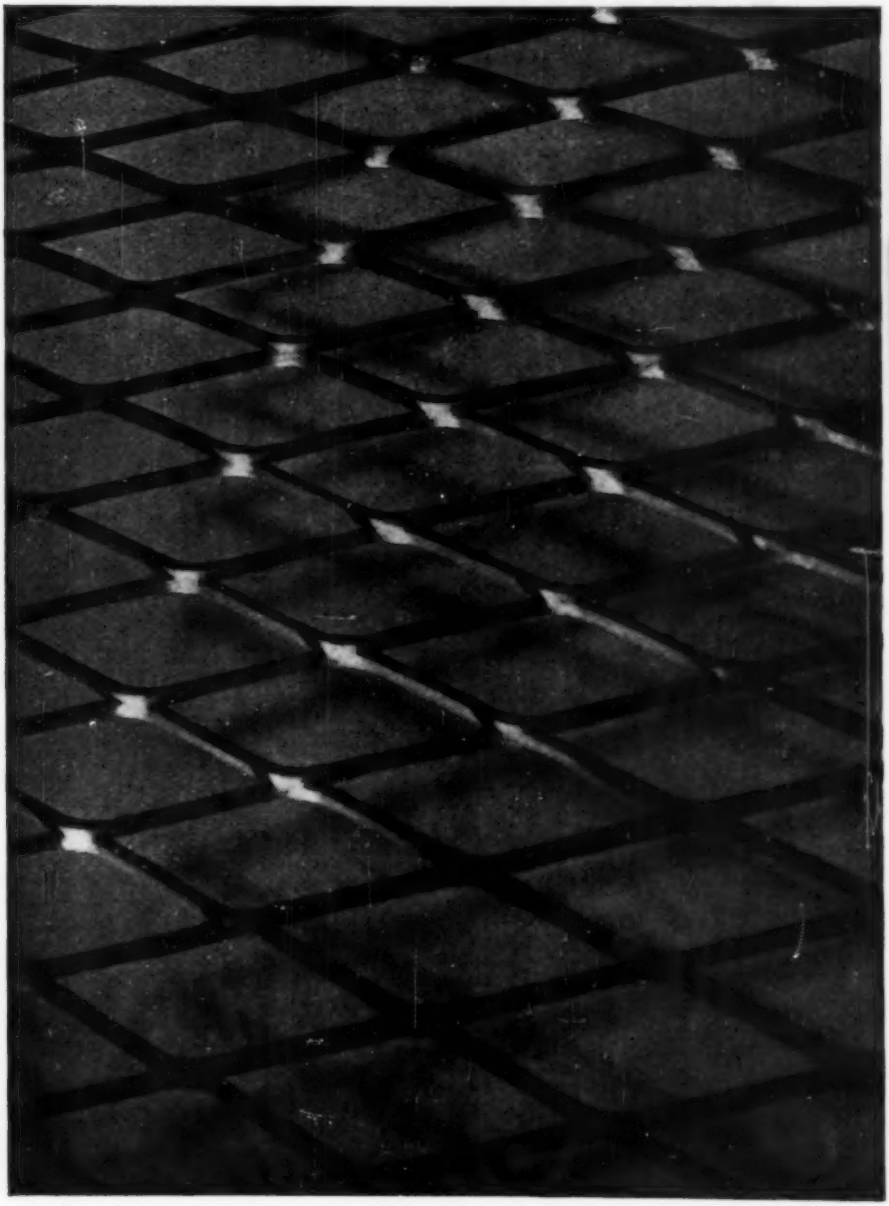

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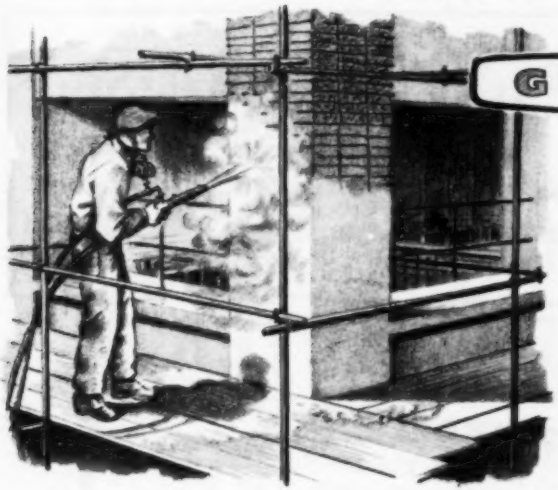

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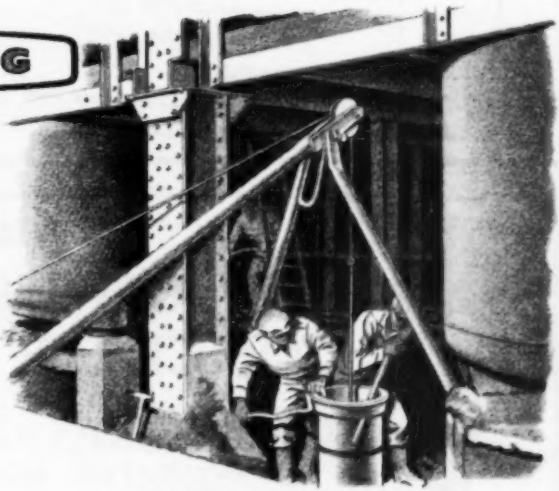

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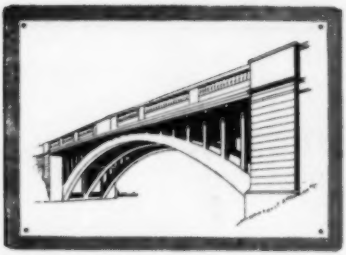
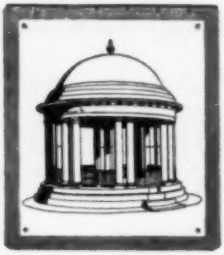
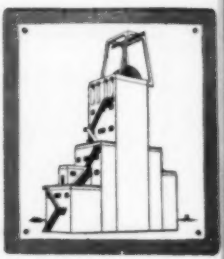
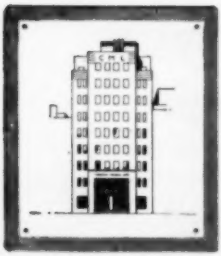
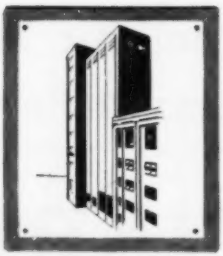
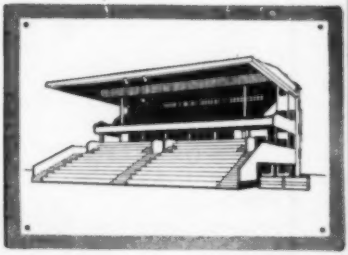




Whatever the Concrete

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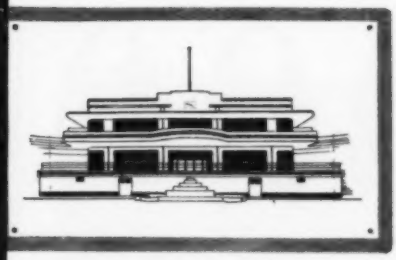

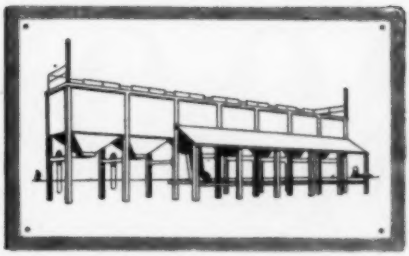

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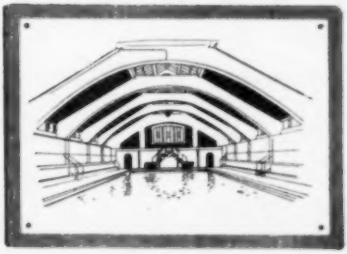

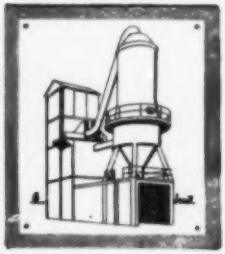

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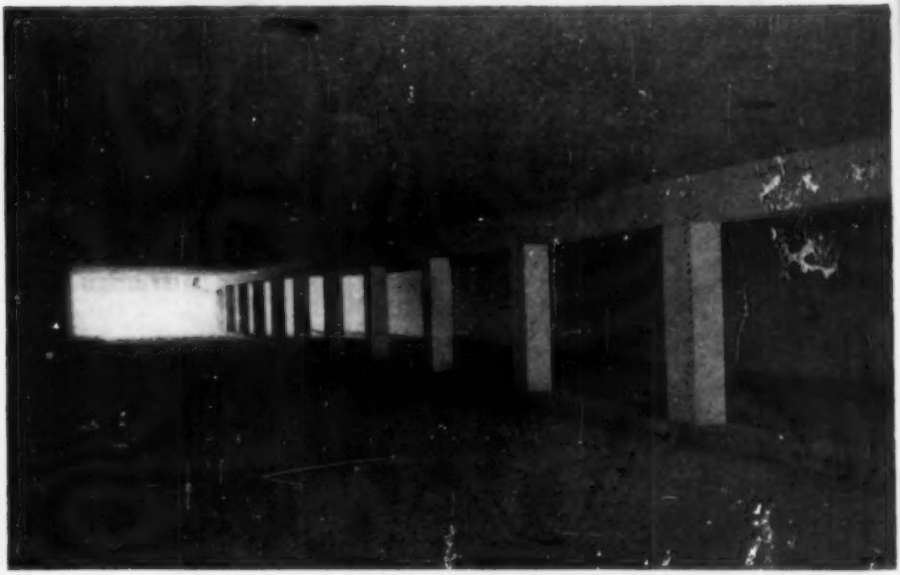

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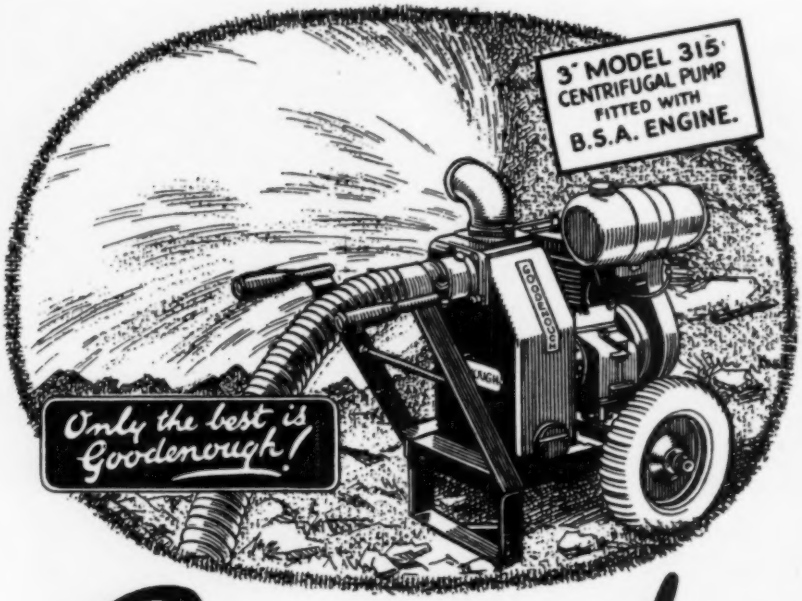

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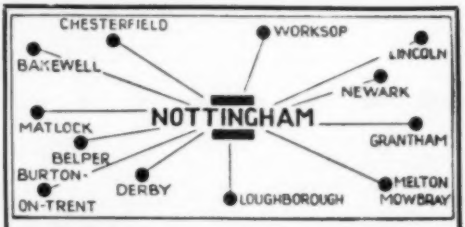

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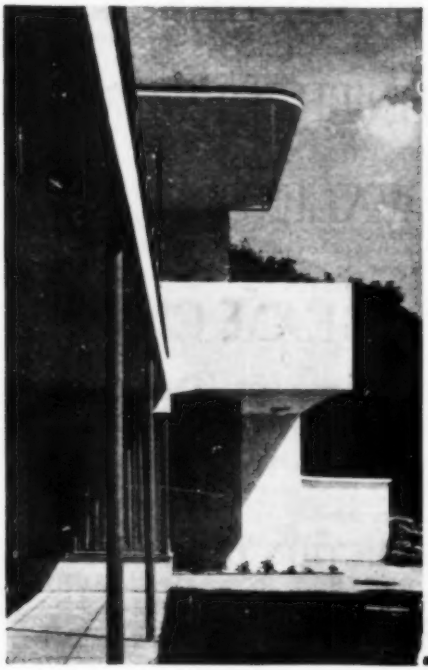

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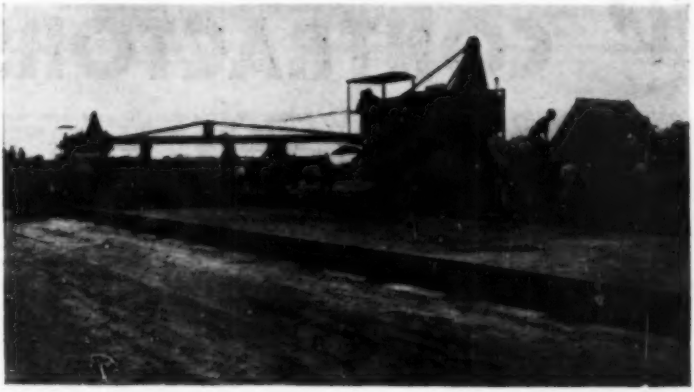

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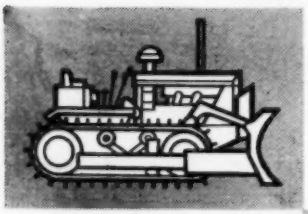

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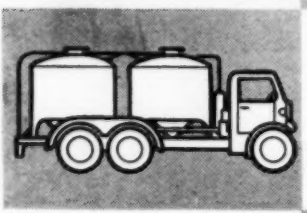

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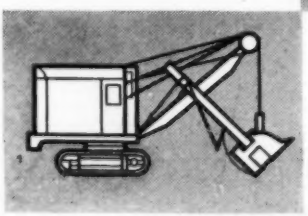

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Volume XLVII. No. 12. LONDON, DECEMBER, 1952

EDITORIAL NOTES

Designations.

THE international and national organisations that are frequently referred to in the press are now so multitudinous that it is difficult to keep pace with them, and the common practice of referring to them by the initial letters of the words comprising their titles makes intelligent reading difficult to all but students of current affairs who have memorised the initials of the titles of these bodies and know what are their objects or purpose. The professions and trades connected with engineering and building are suffering from a similar eruption of new organisations whose members use after their names the initials of the titles of these bodies on all possible occasions. It is a common experience to see designatory letters which are obviously those of a body connected with building but of which one has no knowledge. Some of these organisations, often with high-sounding titles, are formed by astute people who have some understanding of human nature and who know that in most professions and trades there are enough snobbish people who will pay a fee for the use of designatory letters which they think will improve their status among their fellows. In such cases the only qualification for membership is the payment of an entrance fee and an annual subscription. On the other hand, societies may be formed by people who like to preen themselves before their fellows. In the one case a society is formed by one or more people who hope to enrich themselves from the fees and subscriptions, and the members join from snobbish motives. In the other case, snobbery actuates the founders as well as the members.

The desire to acquire the right to use designatory letters after one's name may arise from a wish to appear to be superior to one's neighbours, and an idea that a person who uses designatory letters after his name will command more respect and admiration, and even awe, than one who parades no such letters. How overpowering this snobbishness can be is shown by signatures we have seen followed by the initials of the name of a correspondence school at which a person is taking a course, and of a trade union of which he is a member.

Apart from some of the newer societies whose reputation is doubtful, and which often cater for people in the humbler walks of life but who are themselves not humble, there are the great learned societies and professional associations, membership of which is achieved by examination and is a proof, according to the class of membership, that a member has been trained in his profession or is fully qualified to practise it. In these cases also it is common in Great Britain for the

members to use the designatory letters of the society on their stationery and on all other possible occasions. This habit seems to be more pronounced in Great Britain than in any other country. On the Continent it is rare for a person to indicate the societies to which he belongs, although it is common for him to preface his name by the word engineer, architect, and so on, in order to indicate his profession. Nor is it customary in the United States of America for engineers to indicate on their stationery the societies of which they are members. In Great Britain many engineers and architects expect, and even insist, that the designatory letters of some or all of the societies of which they are members be given after their names whenever they are mentioned in print.

One of the reasons may be that in countries where engineers and architects are permitted to advertise they do not think that there is the same need to mention the societies of which they are members, whereas in a country where they are not permitted to issue public advertisements it may be thought that membership of as many societies as possible and the use of many designatory letters raise their status in the eyes of prospective clients. The societies themselves are sometimes responsible for this practice by requesting their members to use their designatory letters on all possible occasions, and this leads to the curious state of affairs that a society which forbids its members to advertise themselves makes use of its members to advertise the society. In other cases the rules of a society may compel a member to use designatory letters if he wishes to indicate his profession on his notepaper or visiting card. For example, a society which has obtained a charter may stipulate in its rules that a member who describes himself as " chartered " must preface this word with the designatory letters of the class of membership to which he has been elected. The purpose of this rule is no doubt to qualify the description " chartered," because some of the members in some of the classes may not be fully qualified to carry out important work, and the public might be misled if it assumed that a professional man who is entitled to describe himself as chartered is necessarily fully qualified in his profession. As all the members of such a society are chartered, and no one who is not a member may describe himself as chartered, it would be quite redundant to indicate the society to which he belongs if the different classes had not got different professional qualifications. It is, however, doubtful whether the public knows that certain letters before the word " chartered " imply different degrees of qualification in a profession. The rarer use of designatory letters in other countries suggests that members of societies abroad generally have more freedom to indicate their profession in a recognised manner without also indicating a society to which they belong and their class of membership.

The habit of using designatory letters is becoming so wearisome that it has been suggested that they should be used only to indicate that a person has received an honour for merit, and not merely that he has received a reward for political activities or for length of service in a Government department, or because he has passed an examination or has merely paid a fee. It has long been the custom to record honorary degrees only at the time they are conferred and in obituary notices. A similar rule might well apply to membership of learned and professional societies, particularly when a person's qualifications to practise a profession could be better understood by the public by the use of words such as Chartered Civil Engineer, Registered Architect, Chartered Surveyor, and so on, provided that the use of such descriptions were restricted to fully-qualified people.

Design of Rigid-frame Bridges with Kerbs.

By ANDREW GALLIA, A.M.I.E. (Australia).

THE method of design given in the following applies to rigid-frame slab bridges spanning in the direction of the traffic and having two edges stiffened by kerbs and ribs, and assumed to be hinged at the footings (*Fig.* 1). The main features of this type of structure, which has small constructional depth, are a reduction of the bending moment at midspan of the deck slab because of the restraining moments at the corners and because of the stiffening effect of the kerbs considered as structural members, and the provision of vertical abutment walls supported on four sides instead of cantilever retaining walls.

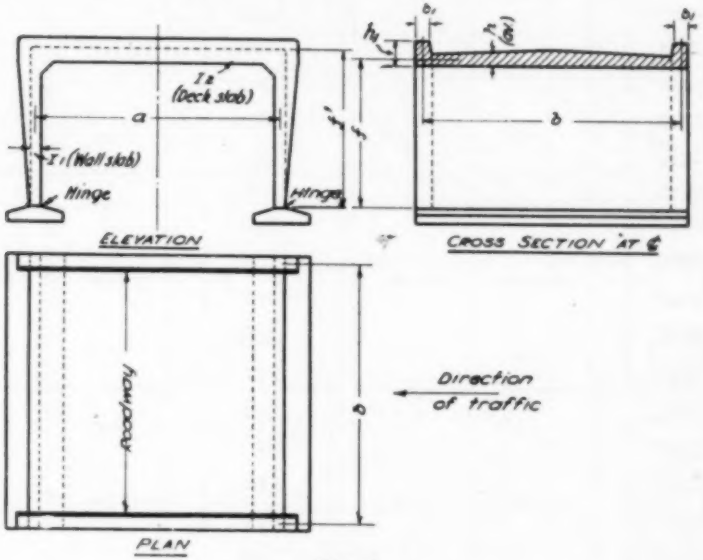

Fig. 1.

The analyses of various investigators * are used as the theoretical basis of the accompanying charts of bending-moment coefficients. The principal notation, in addition to the dimensions in *Fig.* 1, is as follows.

C, bending moment coefficients.
F, line-load per unit length due to loads on the filling behind the wall.
H, stiffness-factor of kerb and deck slab $\left[=\frac{b_1(1-\mu)}{a}\left(\frac{h_1}{h}\right)^3\right]$ or stiffener and wall slab.
K, stiffness-factor of deck and wall slabs $\left[=\frac{I_2 f}{I_1 a}\right]$.
M_F, fixed-end bending moment on the deck slab in a direction parallel to the kerbs. M_{FK}, fixed-end bending moment on the kerbs. M_K, positive bending moment on the kerbs. M_x, positive bending moment on the deck or wall slabs in a direction parallel to the kerbs.

* See bibliography at end of article.

P, concentrated load (assumed to act on a circular area of 1·25 ft. diameter).
p, distributed load per unit area on deck slab.
q, line-load per unit length of kerb.
w, weight per cubic foot of earth.
μ, Poisson's ratio (assumed to be 0·15).

Fig. 2 shows nine possible conditions of support of the deck slab, and *Fig.* 3 shows nine possible conditions of support of the wall slab. The charts in *Figs.* 4 to 6 show the variation of bending moments on the deck slab for the two extreme

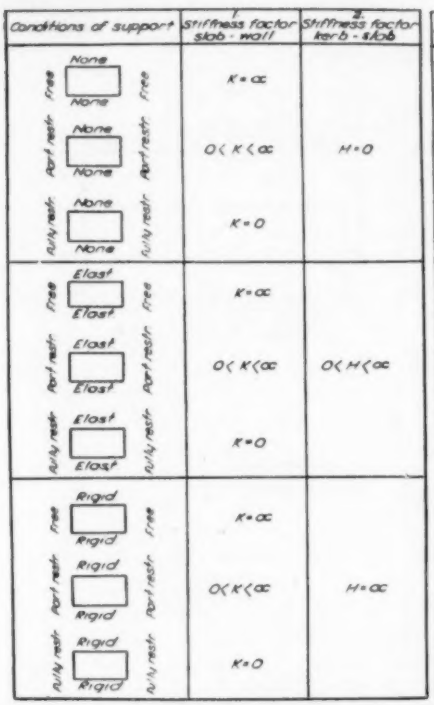

Fig. 2.—Possible Conditions of Support of Deck Slab.

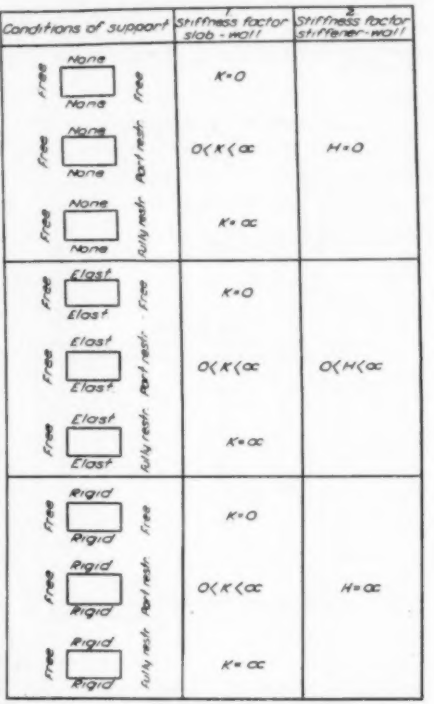

Fig. 3.—Possible Conditions of Support of Wall Slab.

cases of full restraint at the walls ($K = 0$) and free support at the walls ($K = \infty$) for slabs without stiffening kerbs ($H = 0$) and for the limiting case when the kerbs are equivalent to a rigid support ($H = \infty$). The condition of partially-restrained deck slabs is indicated by the curves ($K = 0{\cdot}5$ and $1{\cdot}5$) between those for freely-supported and fully-restrained slabs. The condition of elastic kerbs is indicated by the curves ($H = 0{\cdot}5$) between those for slabs without kerbs and slabs with rigid kerbs.

Fig. 7 gives the coefficients of the fixed-end bending moments, due to earth pressure, on the wall slab and stiffening ribs for values of K from zero (top edge freely supported) to about 1·6. A wall slab without stiffening ribs on the edges

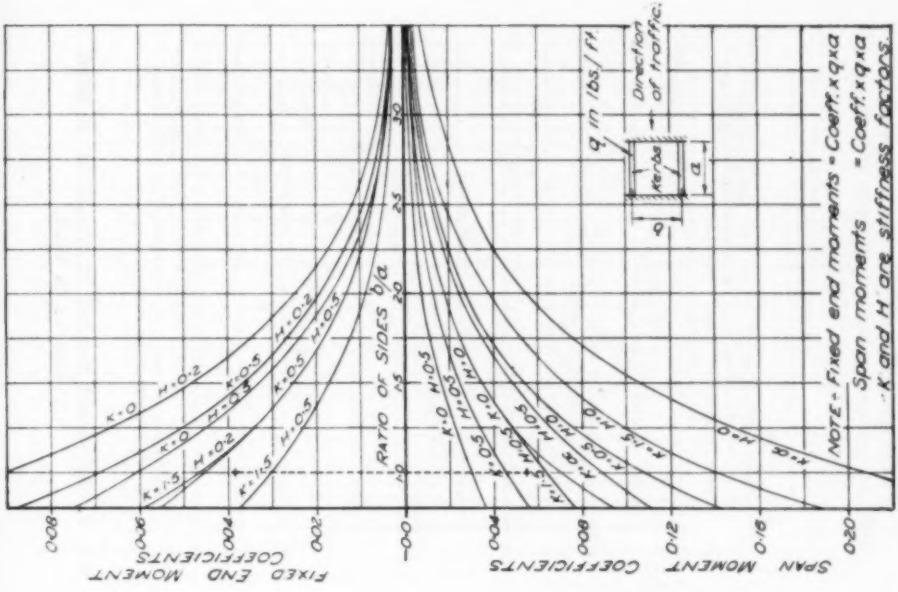

Fig. 5.—Bending Moments on Deck Slab due to Uniform Line Loads.

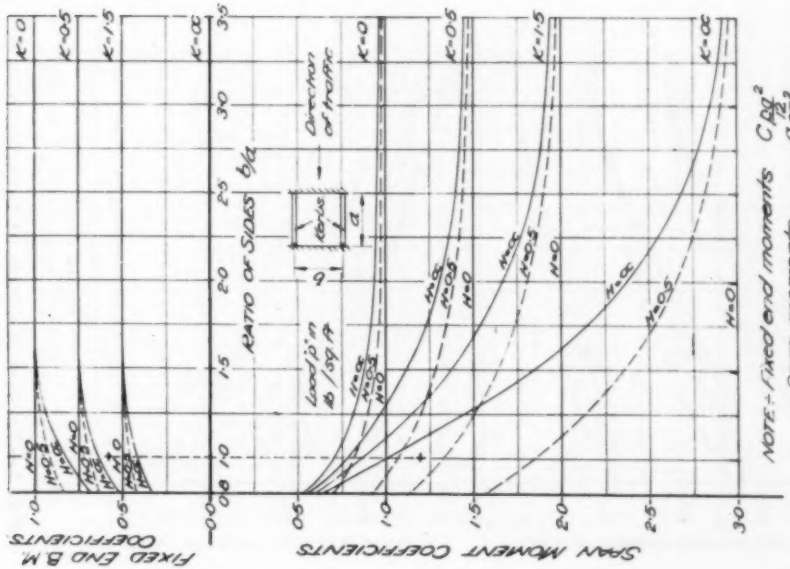

Fig. 4.—Bending Moments on Deck Slab due to Uniformly-distributed Load.

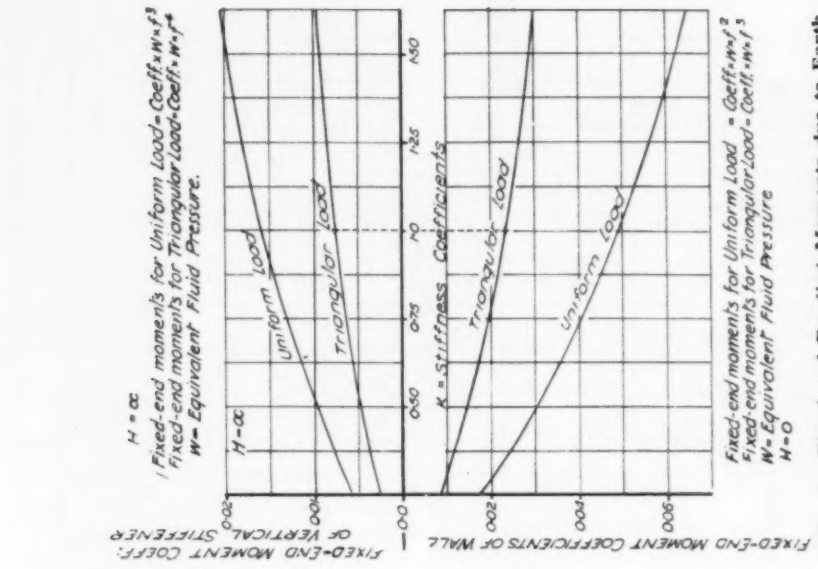

Fig. 7.—Fixed-end Bending Moments due to Earth Pressures on Wall.

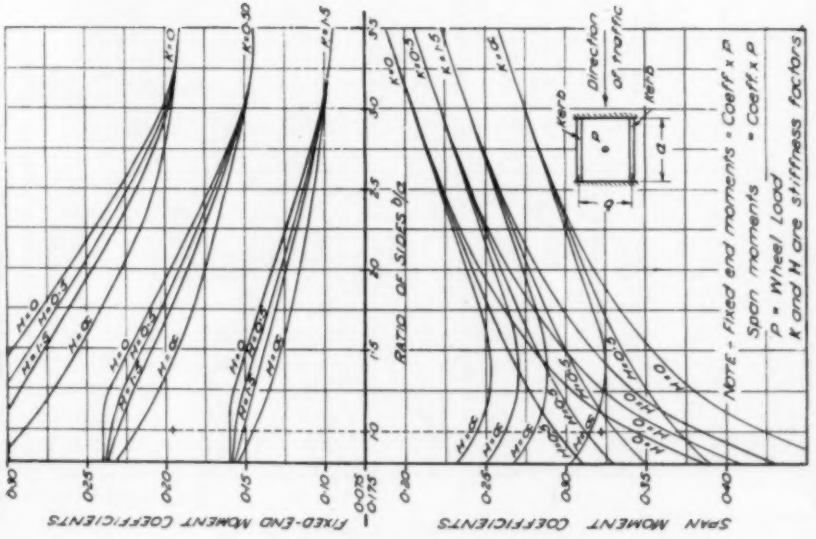

Fig. 6.—Bending Moments on Deck Slab due to One Wheel-load *P* on Slab.

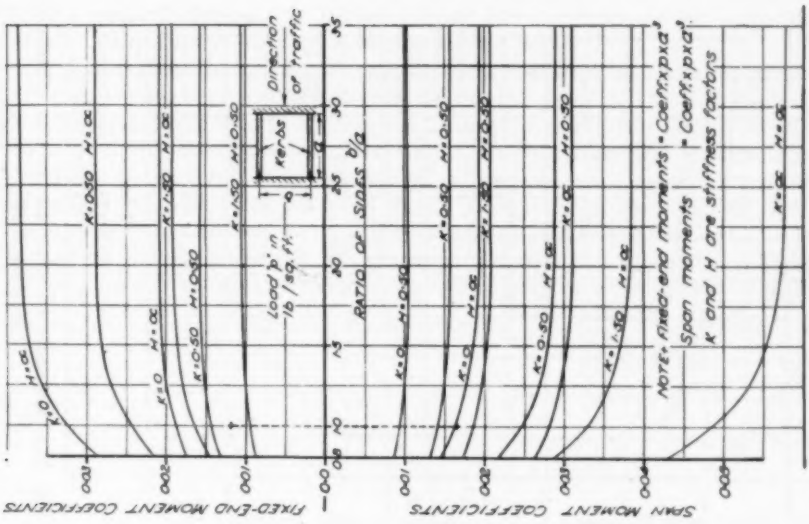
Direction of traffic
Load 'p' in lb / sq. ft.
RATIO OF SIDES b/a
SPAN MOMENT COEFFICIENTS
FIXED-END MOMENT COEFFICIENTS
Note: Fixed-end moments = Coeff.×p.a²
Span moments = Coeff.×p.a²
K and H are stiffness factors

Fig. 9.—Bending Moments in Kerb due to Uniformly-distributed Load on Deck Slab.

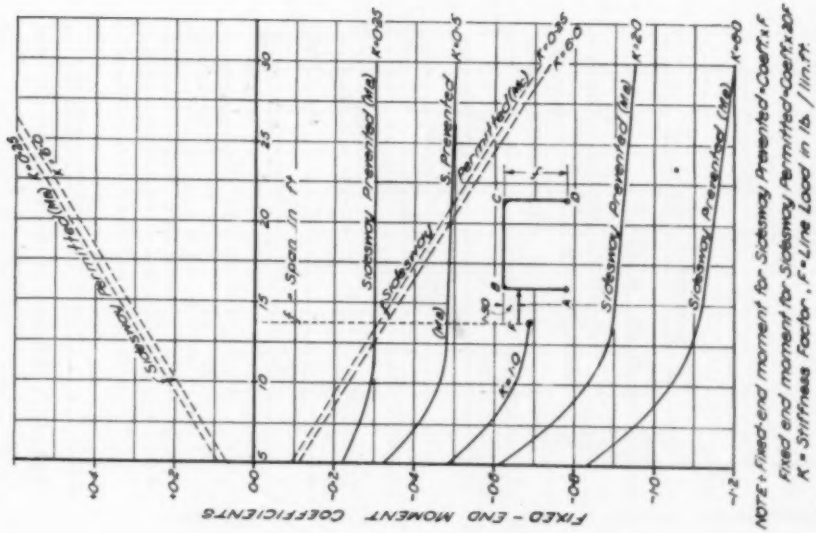
FIXED - END MOMENT COEFFICIENTS
f = Span in ft
Sidesway Prevented (Ma)
Sidesway Permitted (Mb)
Sidesway Prevented (Mc)
Sidesway Prevented (Md)
Note: Fixed-end moment for Sidesway Prevented = Coeff.×F
Fixed end moment for Sidesway Permitted = Coeff.×2Fc
K = Stiffness Factor. F = Line Load in lb. / linear ft.

Fig. 8.—Bending Moments on Wall Slab due to Uniform Line-load F.

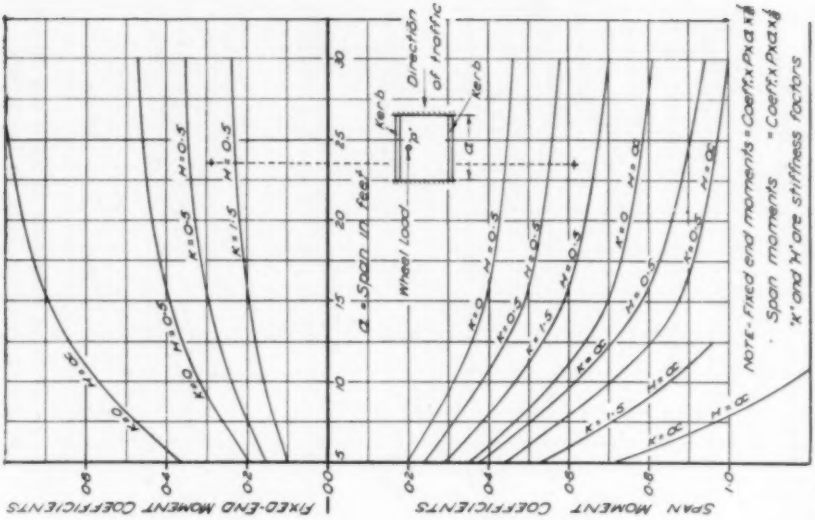

Fig. 11.—Moments on Kerb due to One Wheel-load on Slab.

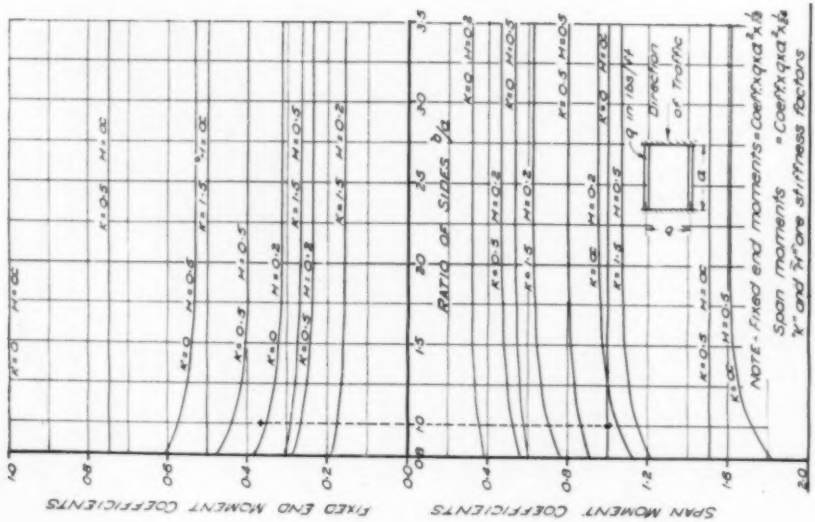
Span moments = Coeff.x qa² x b
'x' and 'H' are stiffness factors
q lbs/ft²
Direction of Traffic
RATIO OF SIDES b/a
SPAN MOMENT COEFFICIENTS
FIXED END MOMENT COEFFICIENTS

Fig. 10.—Moments on Kerb due to Uniform Line-load on Kerb.

is represented by the curves for $H = 0$, and the stiffness of a slab which is rigidly fixed on four edges is represented by the curves for $H = \infty$. The condition for elastic vertical stiffeners is between these two extremes. *Figs.* 9 to 11 give similar data for the bending moments on the kerbs.

Fig. 8 gives the coefficients for the fixed-end bending moments on the wall slab due to a uniform line-load. Curves are not given for the extreme cases of the top edge being fully restrained and the bottom edge freely supported ($K = \infty$) or both edges freely supported ($K = 0$), but the curves for the condition of partial restraint at the top of the wall slab ($K = 0{\cdot}25$, $0{\cdot}5$, $2{\cdot}0$, and $6{\cdot}0$) are given and would be between those for $K = \infty$ and $K = 0$. The two cases of side sway being possible or not are given. The calculation of the line-load due to a load P acting on the surface of the filling behind the wall can be based on Spangler's theory, from which the intensity of horizontal pressure at any point at a depth z below the top of the filling can be expressed by $\frac{1{\cdot}1Px^{1{\cdot}75}z}{(x^2+y^2+z^2)^{2{\cdot}5}}$, where x and y

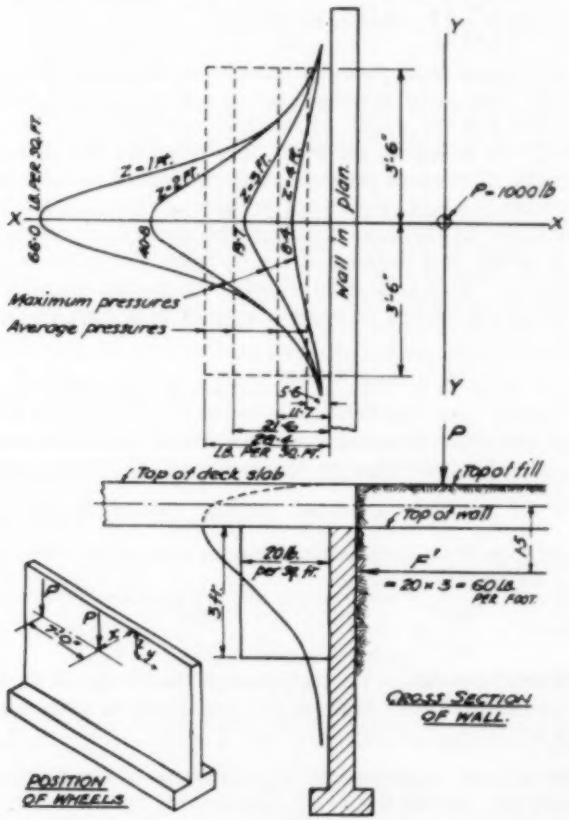

Fig. 12.—Surcharge on Abutment Wall due to Wheel Load (P = 1000 lb.) on Filling.

are the horizontal co-ordinates of the point with relation to the position of the load (*Fig.* 12). The plan in *Fig.* 12 shows the variation of pressure (for $P = 1000$ lb. and $x = 2$ ft.) and the equivalent average pressures (on a width of 7 ft.), from which the diagram of the vertical distribution of average pressure shown in the cross section can be obtained. The total average pressure per foot (F^1) can then be determined, the position of F^1 being approximately at the centroid of the diagram of the vertical distribution.

Example.

The following example shows the method of using the curves for the design of a bridge for which $a = 23{\cdot}5$ ft., $b = 23{\cdot}5$ ft., and $f = 13{\cdot}75$ ft. Assume the average thickness h of the deck slab to be 14·5 in. and of the wall 12 in. Therefore the stiffness-factor K is $\left(\frac{14{\cdot}5}{12}\right)^3\left(\frac{13{\cdot}75}{23{\cdot}5}\right) = 1$. Assume also that the width b_1 of the kerb is 18 in. and its height h_1 is 28·5 in. The stiffness factor H is $\frac{18}{23{\cdot}5 \times 12}(1 - 0{\cdot}15)\left(\frac{28{\cdot}5}{14{\cdot}5}\right)^3$, that is about 0·5.

Loads.—The dead load p is the weight of the deck slab (180 lb.) plus the weight of the 2-in. wearing surface (20 lb.), that is 200 lb. per square foot. The dead line-load q is the weight of the kerb (580 lb.) plus the weight of the parapet (20 lb.), that is 600 lb. per foot. The live load is a 20-tons wheel load (including impact). The earth pressure on the abutment walls is a triangularly-distributed pressure the equivalent fluid pressure w of which is 30 lb. per cubic foot. The surcharge on the ground behind the abutments due to the live load is a line-load F acting at a distance of 1·5 ft. below the centre of the deck slab as shown in *Fig.* 12. Since each main wheel of the 20-tons engine in the Ministry of Transport standard loading imposes a load of 7·34 tons (excluding impact) on the filling,

$$F = \frac{60 \times 7{\cdot}34 \times 2240}{1000} = 1000 \text{ lb. per foot.}$$

Fixed-end Bending Moments on the Deck Slab in the Direction Parallel to the Kerbs.—Due to the uniformly-distributed dead load, the coefficient C of the maximum fixed-end bending moment M_F for $\frac{b}{a} = 1$, $K = 1$, and $H = 0{\cdot}5$ is 0·57 (by interpolation between curves $K = 0{\cdot}5$, $H = 0{\cdot}5$, and $K = 1{\cdot}5$, $H = 0{\cdot}5$ in *Fig.* 4), and

$$M_F = -\frac{Cpa^2}{12} = -\frac{0{\cdot}57 \times 200 \times 23{\cdot}5^2}{12} = -5250 \text{ ft.-lb.}$$

Due to the dead line-load q, the coefficient of the maximum fixed-end bending moment M_F on the deck slab is 0·038 (by interpolation from *Fig.* 5) and

$$M_F = -Cqa = -0{\cdot}038 \times 600 \times 23{\cdot}5 = -535 \text{ ft.-lb.}$$

Due to the 20-tons wheel-load P, the coefficient of the maximum fixed-end bending moment M_F on the deck slab is 0·195 (by interpolation from *Fig.* 6), and

$$M_F = -CP = -0{\cdot}195 \times 20 \times 2240 = -8740 \text{ ft.-lb.}$$

Due to the triangularly-distributed earth pressure, the coefficient of the maximum fixed-end bending moment M_F at the top of the wall slab for $K = 1{\cdot}0$ and $H = 0$ is 0·0235 (from the lower part of *Fig.* 7), and

$$M_F = -Cwf^3 = -0{\cdot}0235 \times 30 \times 13{\cdot}75^3 = -1830 \text{ ft.-lb.}$$

Due to the horizontal line-load F on the wall, the coefficient of the maximum fixed-end bending moment M_F at the top corner of the wall slab for $K = 1$, $f = 13{\cdot}75$ ft., and $F = 1000$ lb. per foot is 0·68 if sides way is prevented (from *Fig.* 8),

$$M_F = -CF = -0{\cdot}68 \times 1000 = -680 \text{ ft.-lb.}$$

The total maximum bending moment ΣM_F at the junction of the deck and wall slabs is, therefore, $-17{,}035$ ft.-lb.

Bending Moments at Midspan of the Deck Slab in the Direction Parallel to the Kerbs.—Due to the uniformly-distributed dead load p, the coefficient of M_x is 1·19 (by interpolation from *Fig.* 4), and

$$M_x = \frac{Cpa^2}{24} = \frac{1{\cdot}19 \times 200 \times 23{\cdot}5^2}{24} = 5450 \text{ ft.-lb.}$$

Due to the dead line-load q, the coefficient of M_x is 0·054 (from *Fig.* 5), and

$$= Cqa = 0{\cdot}054 \times 600 \times 23{\cdot}50 = 7600 \text{ ft.-lb.}$$

Due to the 20-tons wheel load, the coefficient of M_x is 0·322 (from *Fig.* 6), and

$$M_x = CP = 0{\cdot}322 \times 20 \times 2240 = 14{,}450 \text{ ft.-lb.}$$

The total bending moment ΣM_x at midspan of the deck slab is 27,500 ft.-lb. The net maximum positive bending moment on the deck slab is obtained by subtracting the negative bending moment due to the earth pressure from this total, that is the net maximum positive bending moment M_x is $27{,}500 - 1830 = 25{,}670$ ft.-lb., which assumes that the earth pressure is fully active; as this may not be the case, assume the probable maximum positive bending moment to be 26,500 ft.-lb.

Maximum Fixed-end Bending Moments on the Kerb.—Due to the uniformly-distributed dead load p on the deck slab, the coefficient of the maximum fixed-end bending moment M_{FK} on the kerb for $\frac{b}{a} = 1$, $K = 1$, and $H = 0{\cdot}5$ is 0·012 (by interpolation from *Fig.* 9) and

$$M_{FK} = -Cpa^3 = -0{\cdot}012 \times 200 \times 23{\cdot}5^3 = -31{,}100 \text{ ft.-lb.}$$

Due to the dead line-load q, the coefficient of M_{FK} is 0·365 (from *Fig.* 10), and

$$M_{FK} = -\frac{Cqa^2}{12} = -\frac{0{\cdot}365 \times 600 \times 23{\cdot}5^2}{12} = -10{,}100 \text{ ft.-lb.}$$

Due to the 20-tons wheel load, the coefficient of M_{FK} for $a = 23{\cdot}5$ ft. is 0·285 (from *Fig.* 11) and

$$M_{FK} = -\frac{CPa}{8} = -\frac{0{\cdot}285 \times 20 \times 2240 \times 23{\cdot}5}{8} = -37{,}600 \text{ ft.-lb.}$$

Due to the triangularly-distributed earth pressure w on the wall, the approxi-

mate coefficient of the fixed-end bending moment M_{FK} on the vertical stiffener for $K = 1$ is 0·007 (from the upper part of *Fig.* 7) and

$$M_{FK} = -Cwf^4 = -0{\cdot}007 \times 30 \times 13{\cdot}75^4 = -7550 \text{ ft.-lb.}$$

Due to the horizontal line-load F on the wall, when the wheel is on the ground behind the wall, the approximate coefficient of the fixed-end bending moment on the vertical stiffener is 0·68 (from *Fig.* 8), and if it is assumed that the stiffening ribs resist 25 per cent. of the load on the wall,

$$M_{FK} = -\frac{CbF}{4} = -\frac{0{\cdot}68 \times 23{\cdot}5 \times 1000}{4} = -4000 \text{ ft.-lb.}$$

The maximum bending moment at the junction of the rib and kerb is therefore ΣM_{FK}, that is $-90{,}350$ ft.-lb.

BENDING MOMENTS AT MIDSPAN OF THE KERBS.—Due to the uniformly-distributed dead load p on the slab, the coefficient of the maximum positive bending moment M_K for $\frac{b}{a} = 1$, $K = 1$, and $H = 0{\cdot}5$ is 0·016 (by interpolation from *Fig.* 9)

$$M_K = Cpa^2 = 0{\cdot}016 \times 200 \times 23{\cdot}5^2 = 41{,}500 \text{ ft.-lb.}$$

Due to the line-load q, the coefficient of M_K is 1·0 (from *Fig.* 10) and

$$M_K = \frac{Cqa^2}{24} = \frac{1 \times 600 \times 23{\cdot}5}{24}.$$

Due to the 20-tons wheel load, the coefficient of M_K for $a = 23{\cdot}5$ ft., $K = 1{\cdot}0$, and $H = 0{\cdot}5$ is 0·615 (from *Fig.* 11)

$$M_K = \frac{CPa}{8} = \frac{0{\cdot}615 \times 20 \times 2240 \times 23{\cdot}5}{8} = 81{,}000 \text{ ft.-lb.}$$

The total positive bending moment ΣM_K on the kerb is therefore 136,300 ft.-lb., but the net maximum positive bending moment is obtained by subtracting the negative bending moment on the vertical ribs due to earth pressure from this total, that is 136,300 − 4000 = 132,300 ft.-lb.; as before the probable maximum positive bending moment on the kerb may be, say, 134,500 ft.-lb.

Having calculated the bending moments parallel to the direction of the kerbs, the dimensions assumed can be checked and the reinforcement calculated in the usual manner. The influences of changes of temperature, shrinking of concrete, and settlement of foundations are neglected in the foregoing, and no allowance is made for the varying moment of inertia as in practical designs this factor may have little effect.

Transverse Reinforcement.

The transverse bending moments on the deck and wall slabs are not considered in detail. For simply-supported slabs with stiffened kerbs, Mr. V. P. Jensen suggests that the area of transverse reinforcement should be 0·35 per cent. of the cross-sectional area of the concrete. For slabs rigidly fixed at the walls, the amount of transverse reinforcement could be less, but not less than the amount recommended in the British Standard codes of practice or required by the Ministry of Transport standard loading.

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Fly-ash in Concrete for a Dam.

A CONSIDERABLE saving of cement is being effected in the construction of the Hungry-horse dam, Montana, U.S.A., by the use of a siliceous waste product of the combustion of coal in industrial furnaces, known as fly-ash. Research by the Bureau of Reclamation has established that concrete suitable for the heart of a dam can contain 90 lb. of fly-ash and only 188 lb. of Portland cement in 1 cu. yd., the maximum size of the aggregate being 6 in. In the face concrete the quantities are 90 lb. of fly-ash and 282 lb. of Portland cement. The average crushing strengths of cylinders at 90 days exceed 4000 lb. per square inch for the concrete in the dam and 5000 lb. per square inch for the face concrete. The strength at earlier ages is less than that of Portland cement concrete without fly-ash. Concrete containing fly-ash is said to be more workable, less permeable, and less subject to "bleeding" and separation, and to generate less heat of hydration, but the value of fly-ash in counteracting any alkali contained in aggregate is low.

The report of the Bureau of Reclamation states that very fine fly-ash, having a low content of carbon, is the only pozzolanic material which, when used to replace Portland cement, requires less water to produce concrete of a given consistency than does Portland cement without the admixture, in spite of the fact that the specific gravity of the fly-ash is about 2·5, that is only about three-quarters of that of Portland cement; hence the absolute volume of fly-ash is about one-third greater than that of the Portland cement it replaces.

The recommended chemical composition of fly-ash is as follows: SiO_2, not less than 40 per cent.; Al_2O_3, not less than 15 per cent.; MgO, not more than 3 per cent.; SO_3, not more than 3 per cent.; loss on ignition, not more than 10 per cent. Except for carbon content, there is little difference in the chemical composition of fly-ash from different sources. The specific surface (by Blaine apparatus) should be not less than 3000 sq. cm. per gramme, and the residue on a 325-mesh sieve not more than 12 per cent. The analysis of a typical specimen of fly-ash used on the Hungry-horse dam is: SiO_2, 48·2 per cent.; Al_2O_3, 20·1 per cent.; MgO, 1·2 per cent.; SO_3, 2·0 per cent.; loss on ignition, 1·8 per cent.; moisture content, 0·1 per cent.

The fly-ash is delivered in bulk in covered hopper-bottom rail wagons and is transported in lorries to silos at the mixing plants. From the silos to the mixers, the fly-ash travels on belt-conveyors. Due to its fineness and low specific gravity, fly-ash creates more dust than Portland cement. Men working near the material generally wear respirators.

The Strength of Concrete at the Time of Loading.

WE have asked some engineers to comment on the suggestion made in our last number that economy could be made in the use of cement if the allowable compressive stress in concrete were to be based upon its crushing strength at the time when the structure was required to carry the superimposed loads rather than at the commonly specified age of 28 days.

From Mr. W. C. Andrews, O.B.E.

(PAST-PRESIDENT, INSTITUTION OF STRUCTURAL ENGINEERS.)

YOUR proposal to base the allowable compressive stress in concrete upon its crushing strength at the age when it will be required to carry the superimposed loads raises a much larger question—that of design strengths in general. At present, where compliance with codes or by-laws is necessary, it is not permissible to base designs on a specified concrete strength, but to base them on nominal mixtures having certain minimum crushing strengths. While experience shows that the actual strengths obtained from these nominal mixtures vary between very wide limits, in the majority of cases the strengths are between 50 per cent. and 100 per cent. greater than the minimum requirements. This wide range of variation is brought about by a number of factors, not the least of which is the variation in the compressive strengths of cements.

Until it is possible (at least for engineering work) to guarantee more uniform strengths for cements we must still look for average strengths much above actual requirements if we are to ensure that the least strength obtained is not below the minimum specified, and your proposal would suffer the same disability. A first step towards remedying this state of affairs would seem to be a thorough revision of the British Standard for cement.

[It is inevitable that cements from different works have different strengths. If this were not so there would be no progress. The British Standard stipulates minimum requirements only. The amount by which cements exceed these requirements is the criterion of the efforts made by the makers to improve their product. This is to the advantage of the consumer, and will be considered when the standard is next revised. Standards must always lag behind progress. It is also true that cement made at the same works may vary in strength from time to time; this is also inevitable in the present state of knowledge, and is common experience throughout the world. A great deal of research is being undertaken on this problem, and it is to be anticipated that eventually uniformity will be possible.—Ed.]

From Mr. G. A. Gardner, O.B.E.

(CHIEF STRUCTURAL ENGINEER, MINISTRY OF WORKS.)

THE increase in the strength of sound cement mortar with aging is an established thing, and data are available for estimating the lower limit of this increase at, say, three, six, nine, and twelve months after setting. The relationship of this increase to a corresponding increase in various mixtures of concrete can be found, and a reasonably reliable percentage increase established for concrete cubes at these several ages.

Questions will at once arise, however, whether a straight increase in compressive cube strength is accompanied by proportional increases in grip and shear, and whether shrinkage and creep are relevant factors; but these items can be settled, and we may still remain with the result that an all-round increase in the strength of structural members can be relied upon.

Having arrived at this the crucial question would be, How can it be taken into account in design? It would, in many instances, be impracticable to predetermine the date after construction at which full loading would take place, and the contingency of construction loads equal to, or even greater than, the user loads would need to be borne in mind. This being so, it seems to me that the only workable course in the ordinary run of structures would be to take account of any increase of strength with age by a reduction in

the initial factor of safety. This much-abused factor, which has to cover numerous contingent items, could quite properly be reviewed in the above light and due account be taken of the fact that aging would probably rather tend to improve matters. This, I submit, cannot be settled without a close review of the whole question, but I would prognosticate that some worth-while saving in cement would come out of it.

From Dr. W. H. Glanville, C.B.E.

I IMAGINE that there are few, if any, who have been concerned with the preparation of codes of practice for concrete who have not been exercised in their minds about the age at which concrete should be tested and who have not fully realized that the strength of concrete increases with age after 28 days.

Many have had experience of 28-day cube strengths failing to come up to the specified values and have been faced with the problem of deciding what shall be done about it. In such cases the engineer usually looks at the gap between the measured strength and the specified strength to see whether in due course as the concrete ages the strength specified at 28 days is likely to be reached at a later age. If it is he feels happier about matters and acts accordingly. If the design of the structure were based upon the strength at a later age this margin would be lost. Unfortunately existing practice in concrete control is such that it is to be expected that the engineer will be faced with this question at intervals.

Nevertheless, it seems to me quite logical to allow for the increase in strength in cases where adequate control can be exercised, and, on the face of it wrong, in the interests of economy, not to do so. Complaint is often made about having to wait for 28 days before being able to assess the strength of concrete actually placed in the structure, and it is unlikely that tests at greater ages would be acceptable to most engineers. I think, therefore, that the best way of making an allowance would be to retain the 28-day strength as the criterion of quality but to allow an increase of the permissible stresses according to the age at loading. Thus structures loaded at three months might be permitted, say, a 10 per cent. increase in stress and those loaded at one year a 15 per cent. increase.

From Professor G. Magnel.

(UNIVERSITY OF GHENT.)

THE principle of basing designs on the strength of the concrete at the time the structure is to be loaded, rather than at 28 days, is quite sound provided that one makes two calculations, namely one for the structure without the superimposed load, using a working stress based on the crushing strength of the concrete at the time the props are removed, and the other for the structure when it is fully loaded using a working stress based on the crushing strength of the concrete at the age when the live load is applied. On the other hand, in my rather long experience I have never seen a case where a beam or column failed under its working load because the crushing strength was not great enough. This seems to show that the normal working stresses are too low.

However, the tensile resistance of concrete is of the utmost importance in beams. I have seen dozens of cases where beams have cracked under the action of diagonal tensile forces, even when the normal design rules are applied, due to the poor quality of the concrete. Consequently the normal factor of safety against shearing forces is too low. In the first volume of my " Pratique du Calcul du Béton Armé " I explain : (1) That the shear formula (shear stress equals the shear force divided by the lever arm times the width of the rib) is wrong because it assumes that the concrete cracks due to the action of the bending moments ; this results in the actual diagonal stress being 1·15 times the computed stress. (2) That the real diagonal stress is still higher when one crack has occurred due to bending, and that the real stress is then up to 1·70 times the computed stress. (3) In addition, the shrinkage of the concrete causes tensile stresses in the concrete and compressive stresses in the reinforcement. In short, the real diagonal stress is about twice the computed one.

Further, in some countries the stirrups and bent-up bars are not designed to resist the whole of the shearing forces, because the shearing resistance of the concrete is taken into account. This is in my opinion wrong, as the stirrups are not stressed until diagonal cracks occur in the

concrete. Consequently, if designs are based on the strength of concrete at, say, 90 days the permissible shearing stresses must be reduced. This explains why we meet failures due to cracking as a result of excessive shearing stresses, but never failures in bending: the real shearing stresses are twice the computed ones, whereas the bending stresses are smaller than those computed (because Hooke's law is not applicable to concrete).

Another aspect of the problem is that, in my opinion, it is wrong for practical reasons to assess the quality of concrete on its strength at 28 days. This is far too late, and in many cases results in much of the work being finished before it is known whether the concrete is acceptable or not. The criterion should be the strength at seven days, so that there will be an opportunity to alter the mixture if the test results show that this is necessary.

Concrete must be tested on the site at 7 days, that is when there is still time to change the mixture used.

The Belgian regulations specify that the permissible compressive stress adopted in the design of beams must be equal to the crushing strength at seven days divided by a factor having the following values:

Kind of cement	Age in Days at which Load is Applied				
	7	14	28	90	180 and more
Ordinary Portland	4	3·0	2·4	1·9	1·6
High-strength Portland . .	4	3·3	2·8	2·3	2·0
Rapid-hardening Portland . .	4	3·6	3·2	2·7	2·5
Slag Portland .	4	2·8	2·2	1·7	1·4

These figures may have to be slightly altered for British cements. Finally, the proportion and the quality of the cement are not the only factors to be considered in the production of high-strength concrete. Much can be done by careful grading, a low water-cement ratio, consolidation by vibration, and so on. A large maximum size of aggregate also tends towards higher strength, but it is seldom that the spacing of the reinforcement bars permits the use of aggregate larger than $\frac{3}{4}$ in. Discontinuous or gap grading can help, but it is generally uneconomical owing to the waste of part of the aggregate.

From Mr. A. Moller.

(DIRECTOR OF PETER LIND & CO., LTD.)

I THINK that the Editorial Note in your November number sets forth fairly the arguments in favour of using higher concrete stresses, in suitable cases at the discretion of the engineer, when a structure will not be loaded until three months or longer after the concrete has been placed.

I do not, however, expect that very large savings will result because if the effective depth of a slab or the cross-sectional area of a beam is reduced the amount of steel reinforcement must be increased, and there is in all cases a certain cross-sectional area which is the most economical whether the concrete is fully stressed or not. The width of a web of a beam or the thickness of a wall is not normally determined on the basis of the strength of the concrete; more often than not there is a practical minimum below which work becomes uneconomical or unsatisfactory owing to the difficulty of properly placing and ramming the concrete.

For columns of large cross-sectional area I would not hesitate to take advantage of your recommendation.

[The difficulty foreseen by Mr. Moller need not arise if leaner mixtures are used and the cross-sectional areas are not reduced.—Ed.]

Book Review.

"Funções Ortogonais na Resolucão de Problemas da Teoria da Elasticidade." By T. van Langendonc. (San Paulo: Portland Cement Association of Brazil. 1952.)

THIS is the first of two volumes of a book in the Portuguese language dealing with the solution of various problems in the theory of elasticity by the use of orthogonal functions. The general theory is introduced in the first chapter and the remainder of the book is devoted to the application of the method to problems of torsion on sections of various shapes including hollow sections and circular shafts of varying diameters.

The Design of Foundations for Generators.

THE following is an abstract of a paper, by Mr. Paul Rogers, of Chicago, published in the Journal of the American Concrete Institute for November 1951. The author deals only with the work above basement-floor level, below which the work is similar to other heavy

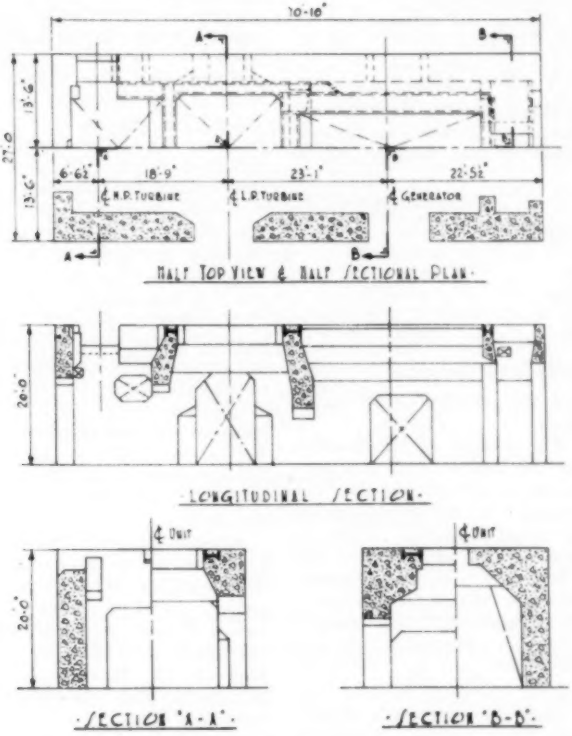


Fig. 1.

foundations. One transverse frame is analysed, but the ordinary computations for beams and columns are omitted.

Fig. 1, which gives the dimensions of an actual turbine foundation, shows that theoretical resonance calculations for such a support are impracticable, if not impossible. Experience has proved that excessive vibration can be eliminated by the simple expedient of maintaining a certain minimum ratio of dead load to live load. The method of design described is applicable to any foundations for turbo-generators of 20,000 kw. or larger. It has been observed that turbine foundations of sufficient mass are free from measurable vibration, and experience has proved that concrete equal at least to two-and-a-half times the weight of all rotating parts should be provided. As the weight of the rotating parts may not be known, it is usual to provide concrete equal to at least one-and-a-half times the total weight of the turbine and the generator. It is important that this ratio be maintained for each frame. The concrete should be so placed that the centres of gravity of the concrete and the machine coincide as nearly as possible. Turbine foundations with very small deflections are very successful. Conse-

quently, it is customary to limit the elastic deformations of the columns and beams to a maximum of 0·02 in. This deformation may be due to bending or axial load or shearing or a combination of these forces. This limitation of deflection, however, does not ensure that there will be no resonance. The relation between mechanical frequency and deflection for an elastically-supported concentrated load is expressed by

Frequency (cycles per minute)

$$= \frac{187{\cdot}7}{\sqrt{\text{deflection (in.)}}}$$

According to this equation, the following deflections should be avoided to prevent resonance under actual loading conditions :

Speed (r.p.m.)	Theoretical deflection for resonant vibrations (in.)
1200	0·0250
1500	0·0150
1800	0·0110
3000	0·0040
3600	0·0028

Loads and Forces.

Fig. 2 shows the position of all major loads and forces.

Load (1). The weight of the turbine is as given by the maker, and is a uniformly-distributed load all around the condenser opening. To this an allowance for impact is usually added. There is wide difference of opinion about the allowance to be made ; in this example 100 per cent. was added for impact. Thus, Load (1) is 190,000 lb. plus 100 per cent. = 380,000 lb., or $\frac{380{,}000}{4 \times 23} = 4200$ lb. per linear foot.

Load (2). The vacuum load of the operating condenser is considered only if an expansion joint is provided at the exhaust flange of the turbine. Alternatively, the weight of the condenser is carried partly by the exhaust flanges and partly by spring supports. In the present case a vacuum load of 400,000 lb. was computed as a uniformly-distributed load, or $\frac{400{,}000}{4 \times 23} = 4300$ lb. per linear foot.

Loads (3) and (4). Concentrated loads of 100,000 lb. and 19,000 lb. respectively were assumed as given by the maker, and 100 per cent. was added for impact.

Loads (5), (6), and (7). Generator loads of 67,000 lb., 100,000 lb., and 67,000 lb. respectively, as given by the maker, were assumed, and 100 per cent. added for impact.

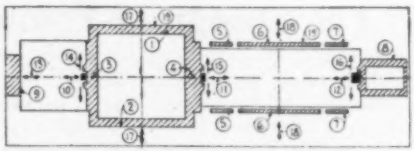


Fig. 2.—Loading Diagram.

Load (8). The weight of the starter was 11,000 lb., and 100 per cent. was added for impact.

Load (9). The high-pressure turbine weighed 120,000 lb., and 100 per cent. was added for impact.

Loads (10), (11), (12), and (13). The horizontal forces were 124,000 lb., 200,000 lb., 117,000 lb., and 60,000 lb. respectively due to thermal expansion. As it is impracticable to provide absolute frictionless movement, horizontal forces equal to the vertical loads times a coefficient of friction result. There is no accepted value of this coefficient of friction, however, and the values recommended by manufacturers range from 25 to 100 per cent. ; a value of 50 per cent. of the weight of all the equipment carried by the cross beams was used in this example.

Loads (14), (15), and (16). Transverse horizontal forces of 180,000 lb., 285,000 lb., and 130,000 lb. respectively, acting in either direction, were assumed. The values of these forces are usually given by the maker, and they originate as follows. Due to the thermal expansion of the generator, the supporting shoes move outward. During a shutdown the plant cools and the shoes move inward. The friction between the shoes and the plates, however, prevents free movement of the shoes, so that the movement may be a different amount for each shoe. After several shutdowns differential movements of the shoes may be so great that the plant may be distorted, causing serious misalignment of the shaft. To prevent misalignment, so-called guide brackets are firmly attached to the beams, and maintain the true position of the shaft even under the most adverse conditions. The forces exerted on the cross-beams by these brackets are considerable, although

they are sometimes computed with extreme conservatism. In addition, these forces are not applied at the centre of gravity of the cross-beams, and in consequence both vertical and horizontal moments act upon the beams. To ensure perfect alignment, the maximum deformation due to these forces and to moments (22) and (23) in *Fig.* 3 is usually limited to 0·01 in. To provide adequate anchorage for the brackets, it is sometimes required that a thick (2 in. or more) steel plate be embedded in the cross-beams, flush with the surface of the concrete. It is possible, and sometimes economical, to combine this steel plate with an embedded steel girder designed to resist any of loads (10), (11), (12), or (13) in *Fig.* 3. The narrow space between the high-pressure and low-pressure turbines, or between the low-pressure turbine and the generator, is seldom adequate to permit the use of a reinforced concrete beam.

Loads (17) and (18). These horizontal forces of 74,000 lb. and 117,000 lb. respectively act in either direction. These are also thermal expansion forces similar to loads (10) to (13). A value of 50 per cent. of the forces due to equipment acting horizontally on the side beams was used.

Load (19). This is a twisting force acting on the shaft and produced by the magnetic reaction of the generator, which tends to retard rotation. The value of this force may be computed by the formula

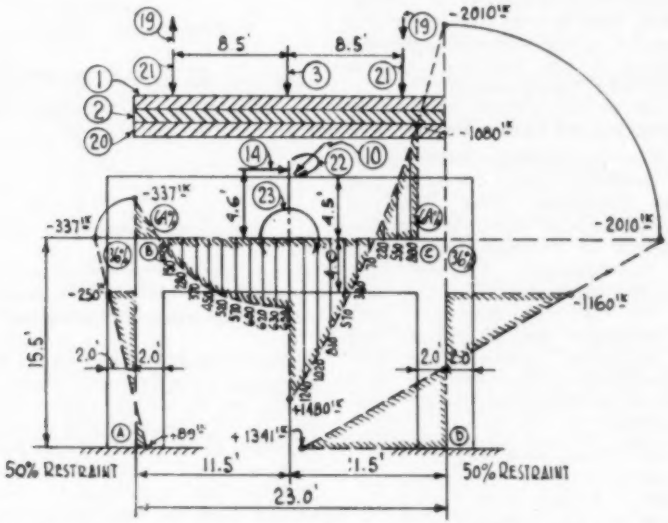
$$\text{Torque (in 1000 ft.-lb.)} = \frac{7{\cdot}04 \times (\text{kw.})}{\text{revolutions per minute}}$$

In the absence of definite data, five times the normal torque is used to provide for shock caused by a short circuit. This torque is resisted by the side anchorage, and the resultant couple produces a downward force at one side and an upward force at the other side, uniformly distributed along the length of the plant. The computed value of the torque load in the present case is 13,000 lb. (*Fig.* 3).

Load (20). The weight of the beam is 3500 lb. per foot.

Load (21). The reaction of each side beam is 94,000 lb.

In addition to these loads and forces, the transverse frame shown in *Fig.* 3 is assumed to be subject to a rise of temperature of 50 deg. F. The relative stiffness



[The symbol " 1k " = 1 foot-kip = 1000 ft.-lb. The bending moments are therefore in thousands of ft.-lb.]

Fig. 3.—Load and Moment Diagrams.

factor of the beam is $\frac{(2{\cdot}75 \times 8{\cdot}5^3)}{(12 \times 23)} = 6{\cdot}12$; the approximate stiffness factor of the columns is $\frac{(10 \times 4^3)}{(12 \times 15{\cdot}5)} = 3{\cdot}42$. The distributing factors are 64 per cent. for the beam and 36 per cent. for the columns. The maximum reaction according to the loading and final moment diagram is 538,000 lb. The bending and axial loads per foot of column are $M = 134{,}100$ ft.-lb. and $N = 53{,}800$ lb. Assuming that $f_s = 16{,}000$ lb. per square inch, $f_c = 975$ lb. per square inch, $n = 10$, $d = 44$ in., $d' = 4$ in., $d'' = 20$ in., and $b = 12$ in., then $NE = 224$ and $KF = 312$, so that compressive reinforcement is not required. Use at least $\frac{1}{2}$ per cent. reinforcement.

Deflection of Cross Beam.

There is no inconsistency between the arbitrary deflection limit of 0·01 in. and resonance deflections. This limit of deflection is insisted upon by turbine manufacturers to eliminate strains and possible misalignment. Applying the moment-area method (*Fig.* 3), only moments on the clear span are used. Positive moments are downward and negative moments upward. The left-hand and right-hand reactions (in thousands of ft.-lb.) of this loading are $R_l = 4940$ and $R_r = 2700$; $x = 9{\cdot}75$ ft.

$M_{max.}$ of moment diagram
$= (4940 \times 9{\cdot}75) - (150 \times 9{\cdot}25)$
$- (280 \times 8{\cdot}25) - (370 \times 7{\cdot}25)$
$- (450 \times 6{\cdot}25) - (520 \times 5{\cdot}25)$
$- (570 \times 4{\cdot}25) - (600 \times 3{\cdot}25)$
$- (620 \times 2{\cdot}25) - (630 \times 1{\cdot}25)$
$- (750 \times 0{\cdot}37) = 48{,}200 - 18{,}760$
$= 29{,}440.$

$$I_{(approx.)} = \frac{2{\cdot}75 \times 8{\cdot}5^3}{12} = 141 \text{ ft.}^4$$

$E = 3 \times 10^6$ lb. per square inch or 432×10^6 lb. per square foot.

Deflection due to bending,

$$\frac{29{,}440 \times 10^3 \times 12}{432 \times 10^6 \times 141} = 0{\cdot}0058 \text{ in.}$$

Deflection due to shear,

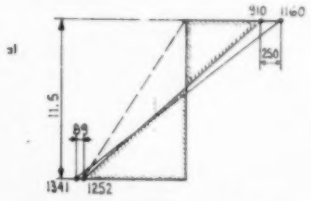
$$\frac{1{\cdot}2 \times 476 \times 19 \times 12 \times 10^3}{8 \times 23{\cdot}4 \times 173 \times 10^6} = 0{\cdot}0040 \text{ in.}$$

Total deflection of cross beam 0·0098 in.

Horizontal Deflection of Frame.

The horizontal deflection of the transverse frame is limited to 0·02 in. The moments acting on the two columns are opposite but not equal; consequently the frame will be subjected to side sway. The resultant moments (clear span only considered) are shown in *Fig.* 4. By the moment-area method the actual deformation can be computed $E = 432 \times 10^6$ lb. per square foot and $I_{(approx.)} = 53$ ft.^{4} The horizontal deflection is therefore

$$\frac{(1252 \times 10^3 \times 5{\cdot}75 \times 7{\cdot}67 - 910 \times 10^3 \times 5{\cdot}75 \times 3{\cdot}82)12}{53 \times 432 \times 10^6} = 0{\cdot}0185 \text{ in.}$$



[Moments are in thousands of ft.-lb.]

Fig. 4.—Moment Diagram for Horizontal Deflection.

The available depth of 21-in. is not adequate for a beam to resist Load (10) of 124,000 lb. within a deflection limit of 0·02 in. Consequently it is advisable to embed a steel girder in the beam. The steel and the concrete beams are not designed as a composite beam. The beams resist the longitudinal and guide-bracket loads only and they are required by the specifications of the makers of the plant. Only in the calculations of the deflections of the steel beams was some part of the concrete included, because the concrete adjacent to the beams would help to resist deformation. It is assumed that the steel girder is 75 per cent. fixed, and that the maximum allowable deflection is 0·02 in. (*Figs.* 5 and 6).

$$I = \frac{P \times L^3}{144 \times E \times d}$$
$$= \frac{124 \times 5{\cdot}83 \times 10^6}{144 \times 30 \times 10^3 \times 0{\cdot}02}$$
$$= 8350 \text{ in.}^4 \text{ (required).}$$

$P = 124{,}000$ lb. $L = 15$ ft. (180 in.). $L^3 = 5{\cdot}83 \times 10^6$ in.^{3}.
$E = 30 \times 10^6$ lb. per square inch.
d (deflection) = 0·02 in.

Actual moment of inertia :

I_0 of web	= 512	in.^{4}
I_0 of flanges	= 45	,,
$Ad^2 = 171 \times 2{\cdot}5 \times 17$	= 7260	,,
36 in. wide concrete transformed section	= 1230	,,
Total	= 9047	,,

$$M = \frac{124{,}000 \times 15}{7} + 18{,}000$$

$$= 284{,}000 \text{ ft.-lb.}$$

$$f_s = \frac{284{,}000 \times 12 \times 10{\cdot}5}{9047}$$

$$= 3800 \text{ lb. per square inch.}$$

Designing Cross Beam for Vertical Moments, Shear, and Torsion.

$M_{max.} = 1{,}480{,}000$ ft.-lb. (*Fig.* 3).

$$d_{\text{required}} = \sqrt{\frac{1480 \times 12{,}000}{161 \times 24}} = 68 \text{ in.};$$

about 86 in. is available.

$$A_{s\,(\text{positive})} = \frac{1480}{1{\cdot}13 \times 86} = 15{\cdot}2 \text{ sq. in.}$$

Use ten bars to provide this total area.

$$v_{max.} = \frac{(538 - 120)10^3}{33 \times 0{\cdot}87 \times 86} = 170 \text{ lb. per square inch.}$$

Use six enclosed stirrups 5 in. apart for the first 3 ft. and 12 in. apart for the remainder. The stirrups are to be welded to the embedded steel girder.

$$A_{s\,(\text{negative})} = \frac{1080}{(1{\cdot}13 \times 86)} = 11{\cdot}1.$$

Use eight bars to provide this total area, bent in from the columns.

The loads acting on the beam do not coincide with the centre of gravity of the beam, so they create torsional stresses. *Fig.* 8 indicates the loads, torsional moments, and stresses, which in this instance are small and can be neglected. The steel girder should have diaphragms between the flanges at close intervals and holes through the web to permit the placing of the concrete.

General Considerations.

The other parts of the turbine foundation, not included in this paper, should be designed similarly to the frame analysed. Torsional stresses, which were negligible in this case, are usually of considerable magnitude in the side beams, and require substantial torsional reinforcement.

All openings should have large haunches to reduce concentrations of stress. The use of cantilevers should be discouraged. The foundation should be entirely separated from the surrounding building construction to avoid transmitting vibrations. Reasonably low stresses should be used in the steel and concrete, and a certain

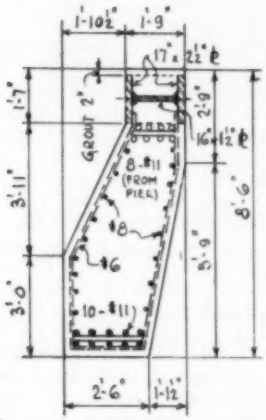


Fig. 5.—Cross Section of Beam between High- and Low-pressure Turbines.

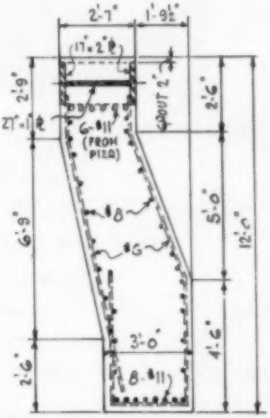


Fig. 6.—Cross Section of Beam between Low-pressure Turbine and Generator.

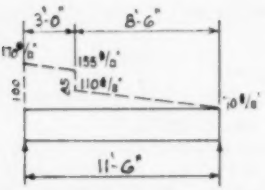


Fig. 7.—Diagram of Shear Forces.

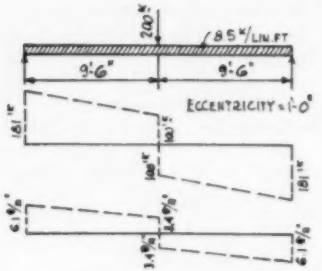


Fig. 8.—Torsional Loads, Moments, and Stresses.

minimum percentage of reinforcement should be maintained.

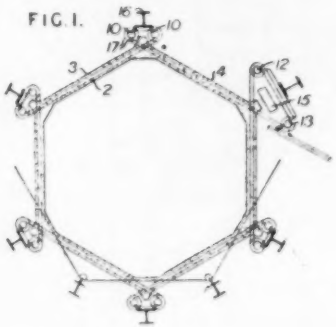
The supporting sub-foundation should be very substantial, preferably a solid slab several feet thick, and adequate dowels should tie the different parts together. Below the so-called foundation there is a sub-basement which houses the condenser, pumps, steam-pipes, intake and discharge tunnels, etc. The quantity of concrete around and below them is so large that it precludes the influence of vibration between the plant and the subsoil. Experience with large generators has led to discontinuing the separation of the turbine and building foundations below the so-called basement level.

To provide a true level for the bearing-plates, a grouting space of 2 in. to 6 in. is usually provided. As laitance on the surface may be objectionable, it is sometimes specified that the concrete shall be thicker and chipped after hardening.

Patent Relating to Concrete.

Prestressed Concrete Structures.

A METHOD of constructing hollow prestressed concrete structures such as tanks and chimneys, comprises arranging a wire to form, in plan, a polygonal figure,



the direction of the wire being reversed at each corner thereof to form a loop projecting beyond the cross section of the intended hollow structure, tensioning the wire at its free ends and possibly at one or more loops, casting the concrete to embed the straight sides of the figure, allowing the structure to harden, and cutting off the projecting loops. Generally the wire will form one or more polygonal-section helices extending the whole or part of the height of the structure. A hollow structure as in *Fig.* 1 is constructed by erecting temporary scaffolding (16) at the corners of the intended structure and looping wires (2, 3) around sheaves (10) attached thereto, the free ends of the wires being secured to a stationary sheave (12) and a movable sheave (13). The wire is tensioned by a hydraulic jack (15) which forces sheaves 12 and 13 apart. Shuttering is then erected and concrete sprayed or cast to embed the straight parts of the wires within the walls (4). After hardening the loops (17) are cut off leaving the shell of the completed structure in compression. In forming cylindrical structures the wires lie tangentially within the walls.—No. 630,591. S. Macerata. July 15, 1947.

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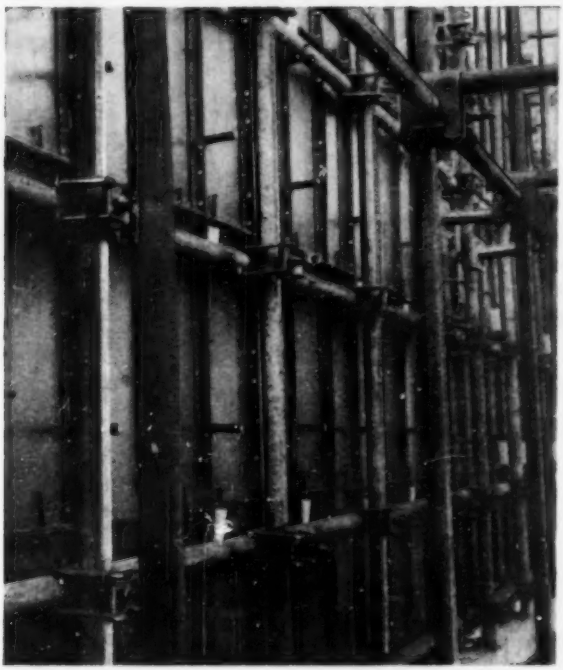

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Diagonal Tension in Reinforced Concrete Beams.

TESTS have recently been made at the United States National Bureau of Standards in Washington on the effect of shearing stresses in reinforced concrete beams. It is known that the capacity of a beam to resist failure by diagonal tension, of which the shearing stress is a measure, is influenced by a number of factors including the compressive strength of the concrete, the amount, distribution, and yield-point strength of the shear reinforcement, the size, number, and yield-point strength of the bars used as tensile reinforcement, and the ratio of the effective depth of the beam to the distance of the load from the support. These four factors were considered in this series of tests.

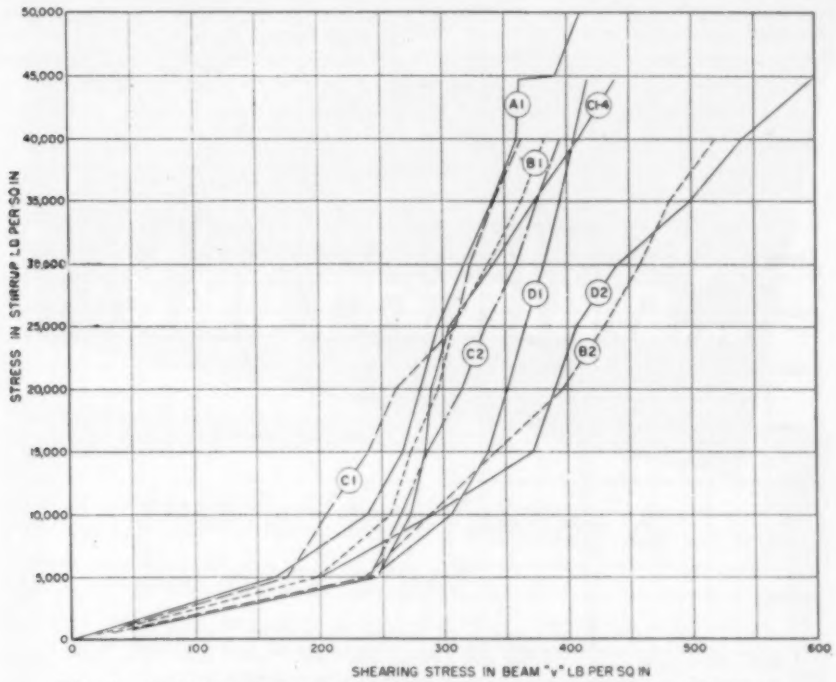

Fig. 1.—Shearing Stress in Concrete and Tensile Stress in Stirrups.

The test specimens were of two sizes, namely 18 in. deep by 8 in. wide for a span of 6 ft., and 15 in. deep by 6 in. wide for spans of 8 ft. to 10 ft., the smaller beams being tested on the longer spans to emphasize the effect of the depth-span ratio. Various amounts of shear and tensile reinforcement were used, the latter having a special anchorage to avoid failure in bond. Generally three specimens were cast for each variation in size, amount of reinforcement, and strength of concrete. The beams were tested in an hydraulic testing machine with the beams supported on rockers. The load was applied at five positions, namely a central concentrated load, two loads 6 in. from the centre of the beam, and at the third and quarter points. In the case of the beams of 9 ft. 7 in. span the loads were applied 2 ft. 8½ in. on each side of the centre. Strains in the bars were measured by electrical resistance gauges and the deflections by dial gauges.

In general the beams developed small

vertical cracks near the centre of the span at a load about 20 per cent. of the maximum load for the beam. At a load of nearly 40 per cent. of the maximum the first crack appeared near the support and this later developed into a diagonal tension crack; as the load was increased this crack gradually extended to a point near the load, and finally opened to produce failure. A stress of 20,000 lb. per square inch in the stirrups was developed at a load between 42 and 58 per cent. of the maximum, and a stress of 40,000 lb.

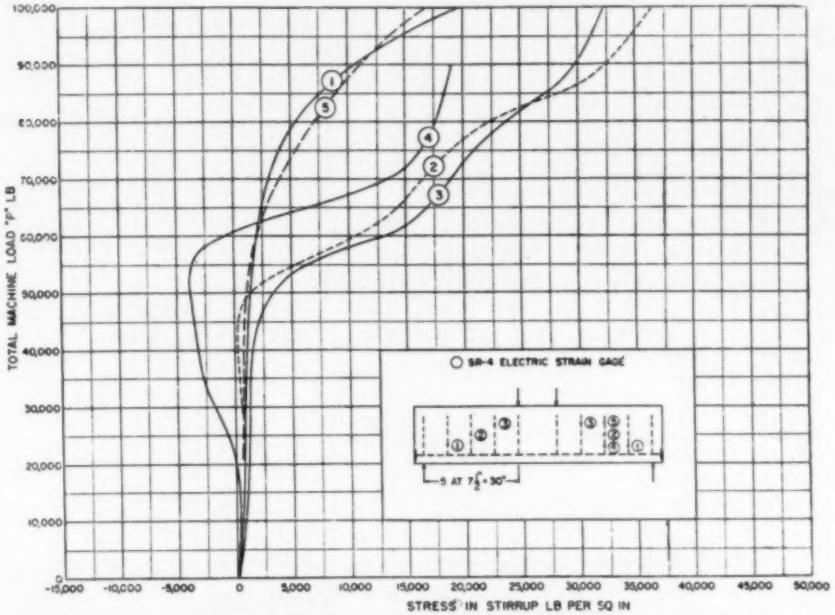

Fig. 2.—Typical Relationship between Stress in Stirrups and Applied Load.

per square inch at loads between 60 and 80 per cent. of the maximum. *Fig.* 1 shows, for each type of loading, the relationship between the computed shearing stress in the concrete and the average stress in the stirrups calculated from the recorded strains. As the yield point was reached in one stirrup the stress in adjacent stirrups increased, indicating that the tension was being resisted by the other stirrups. A typical pattern of the stress variation in the stirrups for increasing load is shown in *Fig.* 2.

As a result of these tests and those of other investigators an expression is given for the maximum shearing stress for the beams of this series. This empirical expression is

$$V_c = 7000p + (0{\cdot}12f'_c)\frac{d}{a} + 2500\sqrt{v},$$

in which V_c is the maximum shearing stress, p the ratio of tensile reinforcement to the cross-sectional area of the beam, d the effective depth of the beam, a the distance from the nearest concentrated load to the support, and v the area of shear reinforcement divided by the product of the width of the beam and the spacing of the stirrups. This expression, it is emphasized, is not of general application, but gives, within the range normally associated with concrete testing, the shear strength of all the beams tested. It shows that the strength in shear varied with the compressive strength of the concrete multiplied by a factor representing the ratio of the depth of the beam to the distance of the load from the support, and with the square root of the ratio of web reinforcement and the first power of the ratio of tensile reinforcement.

Details of these tests, from which the above is taken, are given by Mr. A. P. Clarke in the Journal of the American Concrete Institute for October, 1951.

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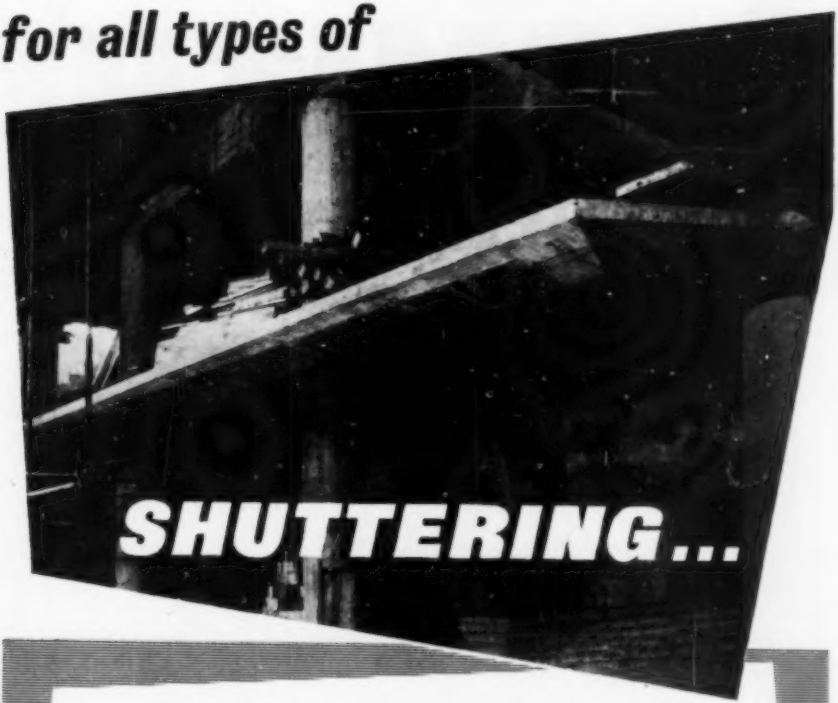

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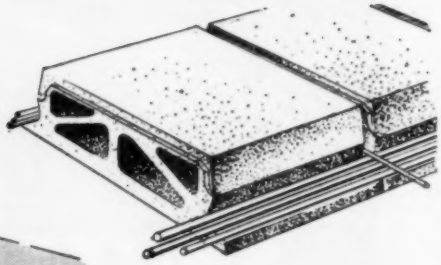

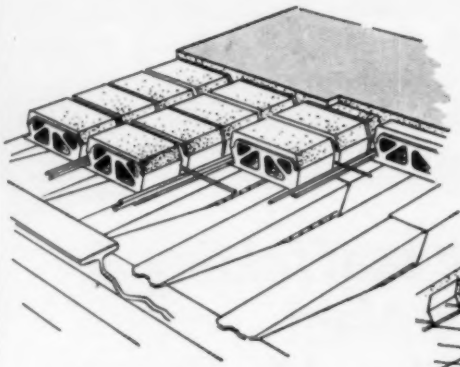

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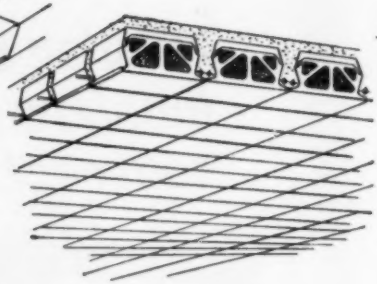

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A Railway Viaduct in Prestressed Concrete.

A PRESTRESSED concrete viaduct has been built by British Railways, Western Region, at Barmouth, North Wales, to replace a timber structure. The viaduct consists of six spans of 40 ft. supported on precast reinforced concrete cross-heads carried by reinforced concrete piers on plain concrete foundations. Each span comprises twelve prestressed precast beams of I section, 2 ft. 2 in. deep by 1 ft. 4 in. wide. On the outer beams on each side is a reinforced concrete plinth extending to 12 in. above rail level. The handrailing is of precast concrete posts with tubular steel handrails. The plinths were cast in short lengths with a gap of ¼ in. between them to avoid their acting in conjunction with the beam (*Fig.* 1).

Fig. 1.

The precast beams, which were made at the Western Region Concrete Depot at Taunton, each contains five cables of the Freyssinet type with twelve 0·2 in. diameter wires in each cable. The cables were stressed to 23 tons each and were grouted under a pressure of 50 lb. per square inch. After erection the beams were stressed in a transverse direction by five cables in each span; the force in these cables is 27 tons per cable. The concrete in the beams is a 1 : 1½ : 3 mixture with a cube crushing strength at 21 days of between 7500 and 8896 lb. per square inch.

The beams were cast in timber moulds and the cavities for the longitudinal cables were formed by inflatable rubber tubes; the holes passing through the lower flange of the beams for the transverse cables were formed by metal tubes inserted in the moulds. The ends of the beams are provided with convex steel plates which fit into concave steel plates set in the cross-heads (*Fig.* 2).

The beams were designed to carry the loads specified in the report of the Bridge Stress Committee (1928) for a maximum speed of 30 miles per hour. One beam in four was loaded at the works at Taunton in the frame shown in *Fig.* 3 by means of hydraulic jacks. At the design load plus an allowance for impact the deflection did not exceed 0·42 in. and under the dead load plus one-and-a-half times the live load and allowance for impact the deflection did not exceed 0·63 in. The computed deflection under working load was 0·5 in. The lower flanges of the beams were examined with a microscope but no cracks were detected under the greatest load.

The beams and cross-heads were erected by cranes travelling on the railway track. The heaviest member lifted by one crane weighed 10 tons. After the twelve beams of one span were erected, mild steel bars with threaded ends were inserted in the cavities left for the transverse cables and were tensioned temporarily by nuts bearing on steel plates. A temporary track was then laid with the sleepers resting directly on the beams and the crane was moved forward to erect the next span. Three spans were erected during a week-end, and a temporary timber span was constructed to join the partly-completed concrete structure to the remaining portion of the timber viaduct so that the railway services could continue during the following week. During this time the transverse cables were inserted, tensioned, and grouted, and the ballasted track laid. The remaining three spans of the bridge were erected in a similar manner a week later. A bituminous compound was applied around the holes to prevent the escape of cement mortar when the cables were grouted. Recesses were formed along the edges of the top flanges of the beams, and after the transverse stressing the joints were waterproofed with hemp and bitumen. Provision for draining the deck is made by 1½ in. diameter steel pipes cast in the plinth.

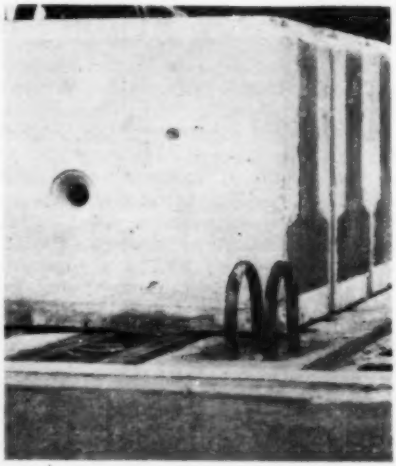

Fig. 2.—Rocker Bearings.

In order to interfere as little as possible with the railway services the reinforced

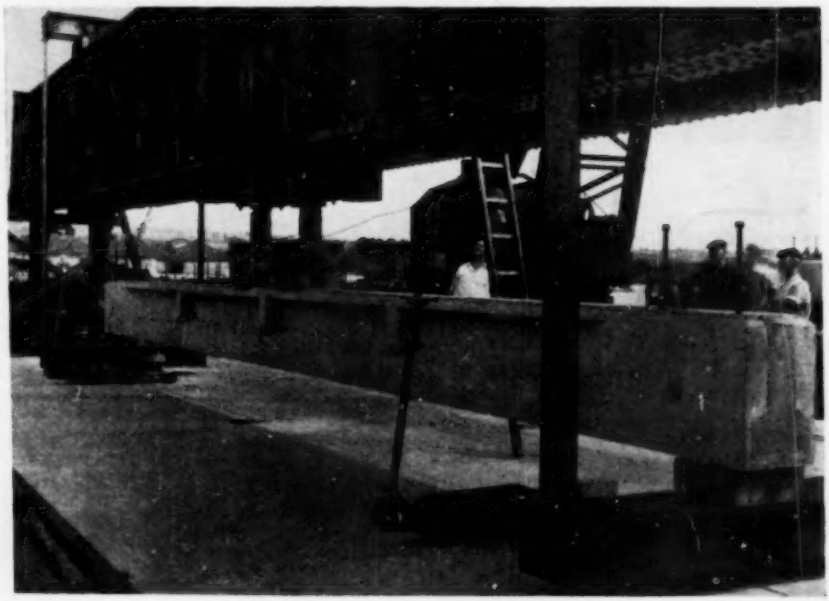

Fig. 3.—Beam and Testing Frame.

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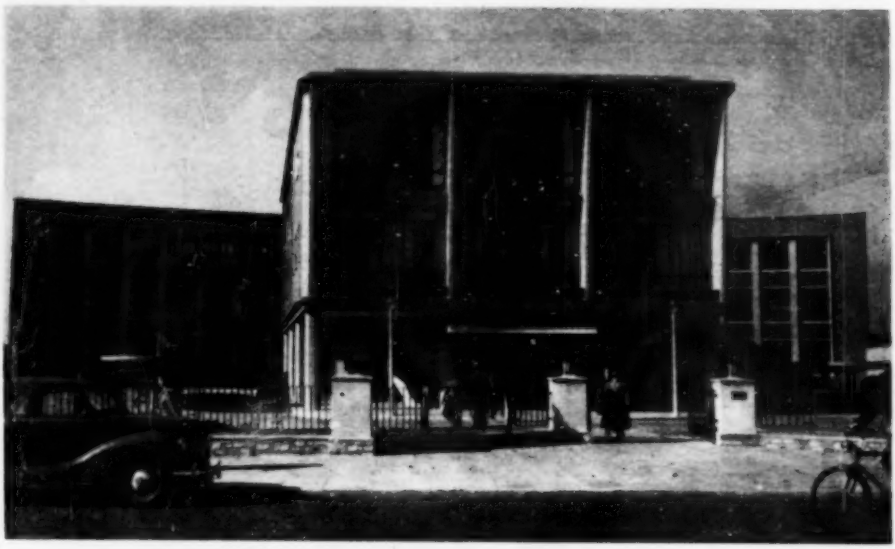

333 Simplex Cast-in-situ Concrete Piles were used in the foundations of the Municipal College Extensions in Anglesey Road, Portsmouth

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concrete piers and mass concrete foundations, which were on firm sand 10 ft. below ground level, were built between the foundations of the timber structure. The pillars are 3 ft. square and faced with precast concrete blocks which, together with the new concrete abutment at one end of the viaduct, were bush-hammered.

The viaduct was designed under the direction of Mr. M. G. R. Smith, Civil Engineer, Western Region, British Railways, and the erection of the cross-heads and superstructure was carried out by the railway staff. The contractors for the reinforced concrete piers and foundations were Messrs. Davies Bros., of Barmouth, and the precast facing blocks for the piers were supplied by the Welshpool Concrete & Construction Co. Ltd.

Hydrophobic Cement.

In the January, 1952, number of this journal reference was made to a report that a Russian chemist had invented a hydrophobic cement. The A.S.P. Chemical Co., Ltd., of Gerrards Cross, now inform us that a hydrophobic cement, for which a patent application has been made, was produced in their laboratory in August, 1950. This type of cement was produced in connection with soil stabilisation in order that the cement could be stored, uncovered, without deterioration. The initial setting time is 2 hours 50 minutes, and the final setting time 3 hours 55 minutes.

Absorption tests were made on prisms made of standard sand, ordinary Portland cement as received from the makers, and with hydrophobic cement fourteen days after casting. The absorptions calculated as a percentage of the dry weight were: Ordinary Portland cement—1 hour, 0·67; 24 hours, 3·22; 3 days, 4·87; 7 days, 6·06. Hydrophobic cement—1 hour, 0·52; 24 hours, 2·63; 3 days, 3·79; 7 days, 4·72.

The moisture-resistant property was tested by spreading hydrophobic cement in a damp cupboard at 90 per cent. relative humidity for seven days and turning it over at intervals. Three 6-in. cubes of 1 : 2 : 4 mixture were then made. The average density was 143 lb. per cubic foot, and the average compressive strength 2390 lb. per square inch at seven days. Similar cubes made with ordinary Portland cement had a density of 147½ lb. per cubic foot and a compressive strength of 2540 lb. per square inch.

It is claimed that this cement is water-resistant so long as it remains static, and the water-resistant properties are reduced when the cement is agitated with the abrasive aggregate in the concrete mixer. It is also shown by tests that this cement imparts a degree of waterproofing to the concrete.

Hydrophobic cement is made by introducing a chemical with the gypsum in the grinding mill. This type of cement is not yet on sale, and further experiments are being made.

Lectures on Concrete.

At Heriot-Watt College, Edinburgh. Mr. John V. Valentine on Concrete Technology. Twelve lectures on Wednesdays starting January 7.

At the College of Building, Clarence Street, Liverpool 3. Mr. J. P. Mitchell on Prestressed Concrete Design; twelve lectures on Mondays starting January 12. Mr. C. A. Perry on British Standards and Codes of Practice for Building Materials; ten weekly lectures on Wednesdays starting January 14.

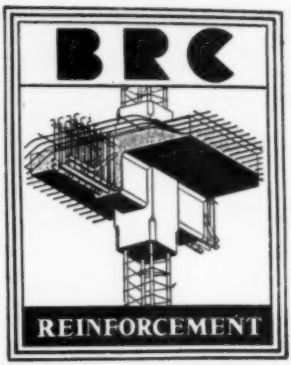
The Departments of Civil and Mechanical Engineering, Nottingham University, with the assistance of the Cement and Concrete Association, have organised a course of lectures on Prestressed Concrete, Design Principles and Methods. The course will consist of eleven weekly lectures on Wednesdays from 3.30 p.m. to 5.30 p.m. commencing January 7. The syllabus and enrolment forms can be obtained from Mr. R. C. Coates, Departments of Civil and Mechanical Engineering, University of Nottingham, University Park, Nottingham.

A New Material for Expansion Joints.

A PLASTIC material for forming a water barrier in joints in concrete is available in the shape of the cross-section shown in *Fig.* 1. The material is claimed to be chemically inert, resistant to water, acids and alkalis, and to withstand variations of temperature between 0 and 212 deg. F. without serious deterioration of its physical properties. The material, which is known as "Durajoint", is supplied in

Fig. 1.

widths of 2¼ in., 3¾ in., 6 in., and 9 in. It is suggested that for these widths the head of water which the joint is required to resist should not exceed, for ordinary quality concrete, 6 ft., 15 ft., 28 ft., and 50 ft. respectively, or twice these heads with high-quality concrete. The material is tough and flexible, and is supplied in lengths up to 50 ft. Where longer lengths are required it is possible to join the strips by overlapping the ends and heating and pressing together the overlaps. The material is manufactured by the Micanite & Insulators Co., Ltd., Walthamstow, London.



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Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATION VACANT. Well-known Midlands firm require for their London branch an engineer to take charge of and control drawing office. Applicants must have knowledge of reinforced concrete frames and steelwork and various forms of floor, roof, and staircase construction; be familiar with drawing office practice (including relevant correspondence) and sitework, and possess experience in the preparation of designs, calculations, and detailing, and have complete knowledge of codes of practice and building regulations. Write fully experience, technical qualifications, and salary required. Box AC76349, SAMSON CLARKS, 57/61 Mortimer Street, London, W.1.

SITUATION VACANT. Designer-draughtsman required in London head office of old-established reinforced concrete engineers specialising in hollow-type floor construction. Progressive post. Write full qualifications and salary required. Box 162, ALLARDYCE PALMER, LTD., 109 Kingsway, London, W.C.2.

SITUATIONS VACANT. The Trussed Concrete Steel Co., Ltd., Truscon House, 35–41 Lower Marsh, London, S.E.1, have vacancies for designers, detailers, and draughtsmen in their London, Birmingham, and Manchester offices. Previous experience in reinforced concrete design and detailing is important. Five-days' week and pension scheme. Apply in writing to the above address, giving full particulars of age, education, and previous employment.

SITUATIONS VACANT. Experienced reinforced concrete designers required for Bristol and Sunbury-on-Thames offices of reinforcement manufacturers. Positions offer opportunity to gain experience of wide range of structures. Attractive salaries offered to suitable applicants. Applications in confidence, stating qualifications and fullest details of experience, to Box 2609, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete detailer-designer and two experienced detailers required by civil engineering contractors. Victoria area. Superannuation scheme. Box 3612, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete designer required for Sheffield office. Write, with details of age, experience, and salary required, to THE SECRETARY, MCCALL & Co. (Sheffield), LTD., P.O. Box 41, Sheffield.

SITUATION VACANT. Technical sales representative required, preferably qualified, for Lee-McCall system of prestressed concrete. Apply to THE SECRETARY, MCCALLS MACALLOY, LTD., P.O. Box 41, Sheffield.

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DRAUGHTSMEN

Simon-Carves Limited recently announced the opening of an additional new drawing office in Mayfair. The firm specialises in the design and erection of Power Stations, Coal Preparation and Chemical Plant, and the new office will be devoted to reinforced concrete work. There are still a large number of vacancies for detail draughtsmen with sound experience of reinforced concrete industrial structures. Salaries are above average, a pension scheme is in operation, working conditions and scope are excellent, and free meal vouchers are provided. Brief relevant details should be sent to

Staff & Training Divn. (Ref. L13),
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All candidates will be interviewed in London.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers and detailers with specialist experience for their Stafford, London, and Newcastle upon Tyne offices. Five-days' week and staff pension scheme. Apply to B.R.C. ENGINEERING CO., LTD., Stafford.

SITUATION VACANT. Consulting engineers require reinforced concrete designer-draughtsman with about 2 years' experience. Salary according to experience. Pensions scheme in existence. Apply in own writing, with copies of references, etc., to J. D. & D. M. WATSON, 18 Queen Anne's Gate, Westminster, S.W.1.

SITUATION VACANT. Old-established Tees-Side firm requires section leader reinforced concrete designer-draughtsman, fully experienced in designing and detailing reinforced concrete structures, foundations, and other civil work. Apply, giving full particulars and experience, quoting reference "D", to Box 3615, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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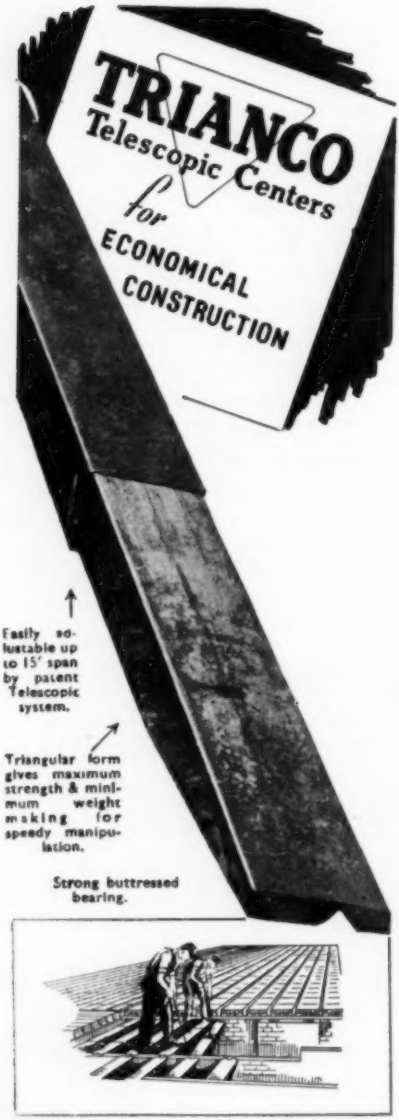

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SITUATIONS VACANT. Design office manager. The Trussed Concrete Steel Co., Ltd., require managers for their local design offices in Manchester and Birmingham. Applications invited in confidence from qualified engineers (degree or A.M.I.C.E. or A.M.I.Struct.E.) with considerable reinforced concrete design experience. Age preferably over 32. Good salary according to experience. Applications giving details of qualifications, present salary, and age should be sent to THE SECRETARY, TRUSCON HOUSE, 35-41 Lower Marsh, London, S.E.1.

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PROFESSIONAL SERVICES. Reinforced concrete engineer requires spare-time work. Fully conversant with L.C.C. regulations and codes. Calculations and full details. Moderate fees. Box 3614, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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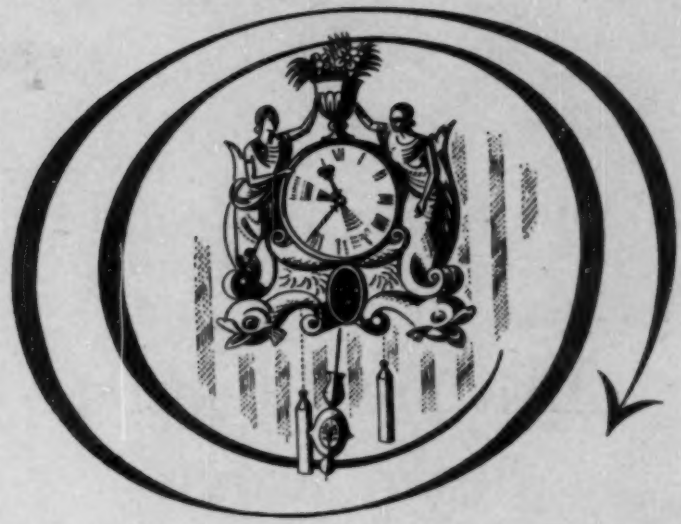

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